UPPER THAMES RIVER CONSERVATION AUTHORITY

FLOOD CONTROL AND WATER CONSERVATION IN THE DRAINAGE BASIN OF THE SOUTH BRANCH OF THE THAMES RIVER

REPORT ON ENGINEERING STUDIES AND COST ESTIMATES

NOVEMBER 1961

VANCE, NEEDLES, BERGENDOFF & SMITH, LIMITED
CONSULTING ENGINEERS
WOODSTOCK ONTARIO

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Chairman and Members,
Upper Thames River Conservation Authority,
Fanshawe Dam,
R.R. # 5,
London, Ontario.

Gentlemen:

We are pleased to submit herewith our engineering report entitled "Flood Control and Water Conservation in the Drainage Basin of the South Branch of the Thames River".

This report generally covers various alternate situations as they apply to dams and channel improvements in the drainage basin and their effect on flood control and water conservation.

We wish to express our appreciation for the opportunity to be of service to your Authority.

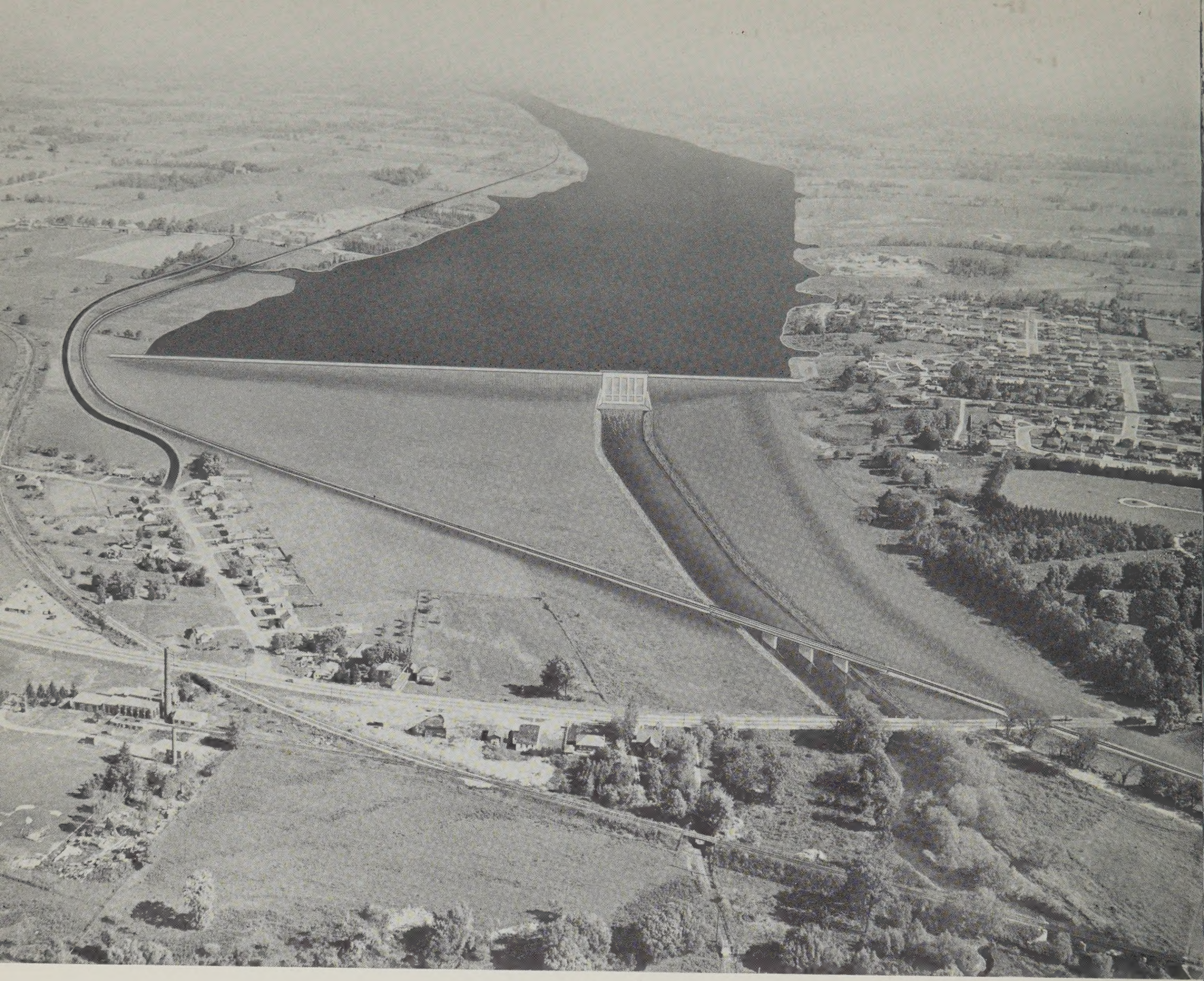
Yours very truly,
Vance, Needles, Bergendoff & Smith Ltd.,

Robert R. Smith

Robert R. Smith P.Eng.
Executive Director

Bernard L. Golding

Bernard L. Golding P.Eng.
Project Engineer.



ALTERNATE I- HIGH DAM ON SOUTH BRANCH THAMES RIVER

VANCE, NEEDLES, BERGENDOFF AND SMITH
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CONSULTING ENGINEERS
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UPPER THAMES RIVER CONSERVATION AUTHORITY

ENGINEERING REPORT

FLOOD CONTROL AND WATER CONSERVATION

IN THE DRAINAGE BASIN

of the

SOUTH BRANCH OF THE THAMES RIVER

VANCE, NEEDLES, BERGENDOFF & SMITH

Consulting Engineers,
Woodstock, Ontario.

October 1961

CONTENTS

SYNOPSIS

This engineering report generally covers two alternate situations as they relate to multi-purpose dams and channel improvements in the South Branch of the Thames River. These alternates are listed and described as:

Alternate 1 - A single high level dam on the South Branch of the Thames River near Woodstock, Ontario. The Construction of this high level structure will involve the relocation of the Canadian Pacific Railway from the south side of the Thames River Valley to the north side and a short channel improvement of a portion of Cedar Creek, so as to provide local flood protection for the City of Woodstock, which would have been provided by a dam on Cedar Creek.

Alternate 2 - A lower level dam on the South Branch of the Thames River near Woodstock, Ontario at approximately the same location as the above, and a low level dam on Cedar Creek, just upstream (south of Highway 401), also in the vicinity of Woodstock, Ontario.

Also described in this report are two channel improvements to the South Branch of the Thames River, near the City of Woodstock, Ontario, which will be made in conjunction with either of the above alternates.

The dams proposed under both alternates and the proposed channel improvements are part of an overall plan for flood control and water conservation in the drainage basin of the South Branch of the Thames River as originally described in the 1952 Upper Thames Valley Conservation Report.

A plan showing the locations of the dams as proposed under each alternate and the proposed channel improvements studied can be found on Drawing No. 1. For purposes of identification, the plan contains stationing in miles, superimposed on the main stem and on all major tributaries of the South Branch of the Thames River. The various sub-drainage basins are delineated. Detailed topographic maps of the reservoir sites are shown on Drawing Nos 2 and 3.

The dams proposed under both alternates are intended to meet a dual purpose of both flood control and water conservation and are multi-purpose dams in the true sense of the word. The same storage volume is to serve several purposes. Separate storage volume at the reservoir sites for each purpose was not available due to the extreme flatness of the land. Normally, the reservoirs under either alternate will be kept full or as full as possible to provide water for purposes of conservation, such as low flow maintenance and irrigation, and for recreational purposes. At certain times, such as the spring of the year or on the approach of a flood-producing storm, the stored water will be discharged and the gates closed or partially closed so that the flood water resulting from snow melt or from the storm on the drainage basin above the reservoir will be contained or limited and the downstream peaks reduced. The various uses of the reservoir proposed under both alternates for conservation purposes and for regulation of flood control are fully described in subsequent sections of this report.

Detailed descriptions of the physical features of the dams, the topography, soils and foundation conditions, etc., for each of the alternates and the proposed channel improvements, as well as the effect of each of the alternates and proposed channel improvements on flood control and water conservation in the drainage basin, are discussed in this report. The advantages of, and detailed cost estimates for each of the alternates and proposed channel improvements are presented together with conclusions and recommendations.

AUTHORIZATION FOR ENGINEERING REPORT

On April 26, 1961, an agreement was entered into between Vance, Needles, Bergendoff & Smith Ltd., Consulting Engineers and the Upper Thames River Conservation Authority. Under the terms of the agreement, Vance, Needles, Bergendoff & Smith Ltd., shall preform certain engineering services in conjunction with the proposed Woodstock Dam and channel improvement at Woodstock, both located on the South Branch of the Thames River and both being intregal parts of an overall flood control and water conservation plan for the Thames River Valley, as described in the 1952 Upper Thames Valley Conservation Report. The principal engineering services to be performed under the agreement are listed as follows:

1. Review in an abbreviated form and verify all surveys and reports already made in connection with the above mentioned projects and complete all additional survey work necessary.

2. Prepare functional design of the dam, make necessary hydraulic and hydrologic studies and cost estimates of the projects.

3. After approval of the design data, functional plans and cost estimates, prepare detailed plans, specifications and tender forms required for the construction of the work together with estimates of costs and assist the Upper Thames Valley Authority in the calling for tender and the awarding of any contracts for the projects.

4. Generally supervise construction of the projects.

Before the first parts of the work, as described in the agreement, were well underway, the Upper Thames River Conservation Authority, at the request of the Woodstock Advisory Board, directed that the engineering work be extended so as to include a study of an additional channel improvement of the South Branch of the Thames River north (upstream) from the Governor's Road Bridge, such work to be done under the original agreement.

The Upper Thames River Conservation Authority, also at the request of the Woodstock Advisory Board, directed that a study be made of the desirability of constructing a high dam on the South Branch of the Thames River at Woodstock with a necessary relocation of the Canadian Pacific Railway Tracks, as an alternate to the low dams on the South Branch of the Thames River and on Cedar Creek, originally described in the 1952 Upper Thames Valley Conservation Report. An agreement to make an engineering study of the high level dam as an alternate to the two low level dams was submitted to the Authority on July 14, 1961 in the form of a letter proposal. This agreement supplemented the original agreement by the Consulting Engineers and was approved by the Authority on August 11, 1961.

SECTION 1 - PHYSICAL CHARACTERISTICS OF THE DRAINAGE BASIN

The drainage basin of the South Branch of the Thames River consists of a mixture of fairly flat and gently rolling heavily farmed land. The highest point in the 280 square mile drainage basin above the junction of the South Branch of the Thames River with the Middle Branch is only 353 feet higher than this junction point. The average slope of the channel of the South Branch of the Thames River is only about 8 feet per mile and is even flatter in the region between Beachville and its junction with the Middle Branch.

The drainage basin is almost completely devoid of woodlands. This fact, when combined with the highly developed system of municipal drains, has removed almost all natural storage in swamps and lowlands, have made the South Branch of the Thames River much more "flashy" than might be expected.

In most of the drainage basin, the actual size of the dry weather channel is quite small. Combined with the very flat slope of the stream, this severely limits its ability to transmit flow; most of the needed capacity in flood times being in the overbank region.

Most of the soils of the drainage basin above Woodstock and of the Cedar Creek drainage basin, which comprise about 47% of the total drainage basin above the junction of the South and Middle Branches of the Thames River, consist of fairly impervious clay type soils of the Perth, Huron and Tavistock series which contribute heavily to runoff and flooding. The remainder of the soils (downstream from Woodstock) are principally pervious sandy type soils of the Guelph and Guelph-Honeywood series.

SECTION 2 - PAST FLOODS

The actual quantitive knowledge of past floods in the drainage basin of the South Branch of the Thames River is quite sparse. Until the turn of the century, no actual measurements of stream flow were made, the actual records or accounts of all floods and flood damages were obtained only from newspaper accounts, diaries or from personal accounts. From that time until the installation of the first recording stream flow gauges in 1959, only stick gauges to measure heights of flow were installed in an attempt to determine discharge. As these stick gauges were in most instances read only once or twice a day, any discharge measured by them, can be considered only an indication of the peak magnitude of past floods, which occurred during this period.

From the information available, a list of all the larger floods which have occurred in the drainage basin, all of which have caused some flood damage or some inconvenience, has been prepared and is given in Table 1 below.

TABLE 1

Floods of Record
South Branch of Thames River

Year	Month
1792	April
1798	April
1868	March
1873	April
1875	April
1883	July
1898	March
1904	March
1926	March
1937	April
1938	February
1945	May
1947	April
1948	March
1954	October
1958	April
1960	March

A comprehensive discussion of these floods and other minor floods, which have occurred prior to 1952, is given in the original Upper Thames Valley Report. Since 1952, three moderate floods have occurred and are the last three listed in Table 1. The flood of October 1954 was the result of Hurricane Hazel, which caused tremendous flooding and flood damage in the Toronto area. Fortunately, only an average of about 3.5 inches in a 48-hour period fell on the drainage basin of the South Branch of the Thames River, much of which was soaked up by the relatively dry soil in the basin at that time, so that no severe flooding occurred. However, there is no meteorological reason why Hurricane Hazel could not have passed over the Thames River drainage basin. A shift of 100 miles in the course of this Hurricane would have caused severe flooding even with the dry soil conditions. The floods of April 1958, and March 1960, were both the result of spring snowmelt or thaw, due to sudden warm weather coincident with some rainfall on frozen ground.

From a study of the floods listed in Table 1 and the other smaller floods which have occurred in this region, there apparently are two separate principal seasons of flooding. These seasons and the principal causes of flooding in these seasons are listed below.

Flood Season	Principal Cause
Winter - Spring (Feb., Mar. and Apr.)	Snowmelt caused by sudden rises in temperature, coupled with rainfall.
Summer - Fall (May through October)	Heavy summer rainfalls and tropical storms.

The approximate peak discharge of some of the major floods of record, occurred at Ingersoll since 1926 as obtained from various sources, are listed in Table 2 below. Only the floods that have occurred since 1959, as recorded by the automatic gauging station located just downstream from the Thames Street Bridge in Ingersoll can be considered as reasonably reliable. Of the recent storms, no record of discharge can be found for the flood of April 1958.

TABLE 2

Peak Discharge of Floods at Ingersoll

Date		Discharge
March	1926	3580 cfs
April	1937	8600 cfs
February	1938	4250 cfs
April	1940	3140 cfs
April	1950	4320 cfs
October	1954	3340 cfs
March	1960	5500 cfs

The floods of July 1883 and April 1937, which were the most severe floods and which had caused the most damage in the drainage basin of the South Branch of the Thames River, were the results of heavy rainfall on saturated ground.

The flood of July 1883 was caused by a severe summer thunderstorm which moved over part of the drainage basin in a wide circle around London and dropped approximately 3 inches of rain on ground already saturated by heavy rain.

The flood of April 1937 was caused by heavy rainfalls which deposited about 4 inches of rainfall in a 48-hour period on ground which already had been heavily saturated by heavy rain and melting snow. However, no snow was on the ground at the time of this flood, nor was the ground frozen.

Generally, the floods which occur in the late winter and spring, due to snowmelt combined with rainfall, although more frequent, are less severe and cause less damage. The flood of April 1947, probably the third severest flood, was of this type.

SECTION 3 - EXISTING FLOOD CONTROL MEASURES

Of the various flood control measures proposed in the original 1952 Upper Thames Valley Report for the South Branch of the Thames River, only a channel improvement between Beachville and Ingersoll has been constructed to date. This channel improvement, which was completed in 1950, was intended principally to protect the quarries in the area between Beachville and Ingersoll and to provide flood protection for the Town of Ingersoll.

The approximate dimensions of the channel improvement as measured in the field at various points along the river between Beachville and Ingersoll are given in Table 3 below. The approximate depths listed in the last column of this table are the approximate depths at which each section of the channel would overflow.

TABLE 3

Dimensions of Existing Ingersoll Channel Improvement

Location	Mile*	Bottom Width (ft)	Side Slope	Approx. Depth (ft)
Ingersoll (Below Thames St. Bridge)	6.4	90	1.75 : 1	16
Ingersoll (Above Thames St. Bridge)	6.4	80	1.75 : 1	14
Ingersoll (Above Mutual St. Bridge)	7.0	80	1.75 : 1	22
Quarry Area	9.4	70	1.75 : 1	25
Beachville	10.8	60	1.75 : 1	18

* See Drawing No. 1 for location

The computed capacities of the channel improvement at each of the above locations are given in Table 4 below, based upon the estimated roughness coefficient of the channel at each point as it exists and based on the assumption that the water surface in the channel is parallel to the channel bottom. No channel constriction, such as bridges was considered.

TABLE 4

Capacity of Existing Ingersoll Channel Improvement

Location	Mile	Capacity (cfs)
Ingersoll	6.4	14,000
Ingersoll	6.4	11,000
Ingersoll	7.0	25,000
Quarry Area	9.4	29,000
Beachville	10.8	14,000

However, the head losses through the Thames Street Bridge, Mutual Street Bridge, Pemberton Street Bridge, and the Canadian Pacific Railway Bridge generally reduce the capacity of the channel in the region upstream from mile 6.4 to about 10,000 cfs due to the slight constriction of the channel at each location. With a very slight overbank flow, the channel in this region can pass about 12,000 cfs with practically no damage.

The channel capacity of the existing channel improvement will be sufficient when the dams proposed under either alternate are constructed, except for that portion of the channel between the start of the channel improvement (mile 5.5) and the tributary joining the river just upstream from the Thames Street Bridge (mile 6.5). This will be discussed briefly in a subsequent section of this report.

SECTION 4 - GENERAL DESIGN CRITERIA

The selection of a design rainfall and resulting design flood for purposes of evaluating the effectiveness of the existing and proposed flood control measures, is a particularly difficult problem. If a small design storm is adopted, any one of several small scale flood control measures may appear effective, and if constructed may give the river valley population a false sense of security. The adoption of an extremely large design storm may result in no flood control measures being constructed at all, as all feasible projects may appear uneconomical. A reasonable compromise between these two extremes has been adopted to provide due consideration of important relevant factors related to costs and benefits.

In drainage basins, composed of residential areas or where the areas to be protected are of high value -- such as the quarry areas between Beachville and Ingersoll, and the Town of Ingersoll and the City of London -- the economies of the situation require that protection against a flood of at least a 100 year frequency be provided. For example, the Soil Conservation Service of the United States Department of Agriculture requires that the minimum flood control storage in small dams be sufficient to contain the hydrograph resulting from a 100 year frequency rainfall, except for outflow through the principal spillway, which must be contained in the downstream channel (either existing or improved) when the dams are located in high damage or high hazard areas.(1)

A design flood resulting from a 100 year frequency rainfall was therefore chosen to evaluate the effectiveness of the different alternate proposals presented in this report, and will be referred to as the Project Design Flood (or Hydrograph). In all cases, a rainfall period of 24 hours duration was selected as the critical time period for the drainage basins involved. The computation of this design (100 year) rainfall, which amounted to 4.5 inches of rainfall over the

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(1) "Limiting Criteria for the Design of Earth Dams," Engineering Memorandum SCS-27, Soil Conservation Service, U.S. Department of Agriculture, Washington 25, D. C., March 1958

entire drainage basin in the 24-hour period, and the resulting Design Flood Hydrographs for each of the principal sub-drainage basins resulting from this design rainfall, are given in Appendix A, of this report. The Design Flood Hydrographs for each of these sub-drainage basins are shown on Figures 25 to 31.

An extensive study was also made of the effect of snowmelt combined with rainfall as a typical flood-providing situation during each spring of the year. This study showed that the combined snowmelt and rainfall, for which an approximate 100-year frequency return period could be assigned, produced a smaller volume of runoff than from rainfall alone and that the time distribution was such that the resultant flood hydrographs produced smaller maximum peak discharges.

SECTION 5 - COMPONENTS AND ENGINEERING FEATURES OF ALTERNATE 1

DESCRIPTION

Under this alternate, a single, high level, rolled earth dam, with a gated concrete spillway section, will be constructed on the South Branch of the Thames River, near Woodstock, Ontario. The details of this proposed structure are shown on Drawing Nos. 5 and 6. As previously mentioned, this high level dam will require that the tracks of the existing main line of the Canadian Pacific Railway be relocated to the north side of the South Branch of the Thames River as they would be flooded out at the proposed full pool elevation of the reservoir. Also, as previously mentioned, a channel improvement for a certain section of Cedar Creek will be necessary so as to provide local flood protection for the City of Woodstock. The Cedar Creek drainage basin and channel improvement, and the relocation of the Canadian Pacific Railway, will be discussed in subsequent sections of this report.

HIGH LEVEL DAM

General:

The proposed dam structure will be located 2300 feet (east) upstream from Highway 19, approximately at the foot of Wellington Street. This location, which is upstream from the dam site proposed under Alternate 2, was dictated by the grade requirements of the proposed relocated Canadian Pacific Railway, as will be described in later paragraphs.

Under this alternate, the proposed maximum water surface of the reservoir created by the dam was set at Elevation 950.0. This is the highest allowable elevation which would still permit successful operation of existing municipal drains tributary to the South Branch of the Thames River, and which would not cause extensive extra flooding of valuable farm land. With the water surface at this elevation, the useable water storage volume created by the dam will be 13,385 acre feet. An additional 651 acre feet will exist as permanent dead storage below the crest of the dam, which is not considered useable except to provide a silt storage area. When the water surface is at this maximum elevation of 950.0, a land area of 1,131 acres will be flooded. Storage-elevation and area-elevation curves for this proposed reservoir are shown on Drawing No. 6. A topographic map showing the location of the proposed dam site and the general reservoir area are shown on Drawing No. 2.

Access to the proposed dam and machinery deck will be from the Woodstock (south) side of the South Branch of the Thames River and an access road will occupy the dedicated, but as yet unused, right of way of Rivercrest Drive. No grade-crossing of the Canadian Pacific Railway Tracks will be necessary in this area, as the railroad will be moved to the other side of the river as previously mentioned.

A parking lot will also be constructed on this side of the valley to provide necessary parking space for the employees and visitors. Both the proposed access road and parking lot are shown on Drawing No. 5. A small operations and administration building will also be constructed on the south side of the valley in the vicinity

of the parking lot. The final location of the operations house and administration building as well as the details of the structure will be determined during the preparation of final plans if the alternate is adopted.

Soils and Foundation Conditions:

In order to determine foundation conditions at the proposed dam site, a preliminary soils exploration and rock drilling program along with hydraulic pressure testing of the underlying rock, was undertaken. The soils exploration and drilling program consisted of placing 12 cased hole borings through the soil overburden to bedrock and then coring the bedrock for depths of 30 to 50 feet. This was followed by hydraulic pressure testing of the rock in certain holes to discover any seams which would transmit water under pressure. The soils and rock profile along the centerline of the proposed dam, as determined by the soils exploration and rock drilling program, is shown on Drawing No. 4. All soils were classified in accordance with the United Soil Classification System. (1)

The soils at the dam site were found to consist of glacial till and alluvium deposits, overlying limestone bedrock at a shallow depth. The limestone bedrock is encountered at depths ranging from 10 feet in the stream flood plain to 55 feet at the valley walls.

The glacial soils are located in the valley walls on each side of the flood plain. They are quite variable in type and density, and consist predominantly of sands and gravel with varying amounts of silt and clayey silt (SM, SC, GW, GP and GC). Numerous boulders were encountered in the north valley wall.

The alluvial soils are located in the flood plain and in a river terrace that rises about 20 feet above the flood plain along the south valley wall. The alluvium deposits consist of clean sand with a few areas of silt and silty sand (SP and SM).

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(1) "The United Soil Classification System" U. S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi. March 1953 (Revised April 1960)

The limestone bedrock is of the Norfolk formation. No seepage cavities were encountered in the boring explorations at the site, nor are any known to exist in the area. However, the bedrock was found to be extensively fractured. Rock core lengths were from one inch or less, up to six inches. Several seams or zones of highly fractured rock were noted. Hydraulic pressure tests conducted on the bedrock in the drill holes showed that high rates of horizontal flow of water could be expected in portions of the top 20 to 30 feet of the bedrock. However, the tests indicated vertical flow to be of a considerably smaller magnitude. Below 20 to 30 feet, the bedrock was found to be generally tight to a depth of 50 feet which was the bottom of the rock exploration.

Generally, the soils exploration and rock drilling program undertaken, demonstrated that the foundation conditions for a dam at this site are reasonably good, with the exception that the area is generally subject to seepage, which is treated in subsequent paragraphs. Bedrock, having only a thin upper surface of weathered material, was found to be approximately at Elevation 910.0 at the center of the valley, thus providing excellent support for the concrete overflow spillway section quite close to the surface and thereby keeping the amount of concrete in the spillway section to a minimum. In order to provide a better foundation for the ogee section and the footings of the training walls, the top layer of soft weathered rock will be removed in these areas.

Since, as previously mentioned, the rock underlying this dam site is extensively fractured, it will be necessary to grout the bedrock for an average depth of about 38 feet under the full length of both the embankment and spillway region. The grouting will be done through a concrete grout cap, 8 feet wide by 3 feet deep, underneath the rolled earth embankment sections and through the first pour of the concrete overflow (ogee) spillway section (probably 4 feet in depth).

Generally, both the glacial till and alluvial type soils comprising the overburden above the bedrock will be satisfactory as foundation material for the earth embankment and no special treatment for settlement or stability will be required. To prevent seepage through this region, an impervious cutoff trench to bedrock under the full length of the earth embankments on each side of the spillway will be necessary. The impervious material required for

backfilling the cutoff trench is readily available as sandy-clay (SC) or gravel-sand-clay mixtures (GC) in adjacent glacial till deposits.

The bottom width of this impervious cutoff trench was set at 20 feet to enable excavating and compacting equipment to operate efficiently. Dewatering of the excavation for the cutoff trench by well points will undoubtly be necessary.

The borings also indicated three other possible areas of seepage loss:

1. The first other possible area of seepage is through a permeable sand, gravel and boulder strata in the overburden on top of the bedrock, under and beyond the north abutment to the north of the impervious cutoff trench.

   The exploratory boring data show a layer of clay and silty sand covering the ground in this area, which should act as a blanket and minimize seepage. Since there is a good indication that seepage through this stratum will not be excessive, no special treatment will be provided during the construction phase, if Alternate 1 is adopted. Should the seepage prove to be excessive after the dam is in operation, it can be treated by grouting, by the placement of a clay blanket, or by chemical treatment. Although the cost of any treatment at a later date may be slightly greater than during the dam construction, it is considered advisable to provide treatment only if proven necessary.

2. It is possible that some seepage losses could occur beyond the south abutment around the cutoff wall of the dam in the alluvial deposits noted there. For two reasons, it is proposed that this area be treated only if found to be necessary after construction. First, the area of the alluvium deposit appears to be small and the length of flow will be fairly long, indicating the resulting seepage losses will be low. Also, the existence of springs just above and to the south of this area indicates that underground seepage is restricted in this area.

3. The third possible area of seepage loss is within the bedrock beyond the grouted areas. This condition is indicated by the high horizontal flows measured at some points within the rock by the previously mentioned pressure tests. The relatively high resistance of the bedrock to vertical flow and the long flow distance to circumvent the grout curtain should reduce the seepage considerably in this area. However, should the seepage still be excessive, the grout curtain can be extended into the abutments of the dam at a later date.

Embankment Section:

The proposed rolled earth sections of the dam are to be of the zoned embankment type, with an impervious central core wall to prevent seepage. This impervious core will be surrounded on each side by a shell of free draining material of high inherent stability which will enclose and support the less stable core. (See Drawing No. 5.) Satisfactory material for the impervious core is available as sandy-clay (SC) and gravel sand clay (GC) in glacial till deposits immediately adjacent to the dam site. Free draining sand and gravel material for the shell is available in adjacent sand and gravel pits in the vicinity of the dam in the form of well graded sands (SW) or gravelly sands (GP). There is also a possibility that some of the silty sand material (SM or SP) excavated from the cutoff trench and spillway could satisfactorily be used for the shell material. However, because of the great variability of this material and the fairly large percentage of fines contained by it, the decision was made to roll this material directly into the proposed railroad embankment, where it would immediately be needed, and to bring in the slightly better graded gravel-sand (SW and GP) material for the shell from the adjacent sand and gravel pits. This procedure also saves the cost of stockpiling and rehandling any portion of the excavated material which might be useable.

After much study of the shell material that would be used in construction, a slope of 2-1/4 to 1, was selected for both the upstream and downstream faces of the dam proposed under this alternate, for purposes of this report.

The 10-foot width of the top of the impervious core was the minimum dimension which will permit economical placement

and compaction of the impervious embankment material. The side slopes of the impervious core were determined by the criterion (applied at the maximum section) that the thickness of the core at any elevation cannot be less than the height of the embankment at that elevation.

The proposed embankment will have a crest width of 18 feet and a freeboard of 8 feet (top of the embankment at Elevation 958.0) in conformance with the recommendations and formula of the United States Bureau of Reclamation. [1] Both the upstream and downstream faces of this proposed structure will be protected by rip rap. The rip rap on the upstream face will extend to the top of the dam as a protection against wave erosion. The rip rap on the downstream face will extend from the ground to Elevation 935.0, to protect the embankment from possible erosion due to an extremely high tailwater condition. The remainder of the downstream face will be sodded. A one foot thick gravel sand layer or blanket (GW classification) will be provided under the rip rap slope protection to prevent fines from being washed out through the rip rap by wave action and to insure that the phreatic line stays within the toe of the embankment. A toe drain will be installed along the downstream toe of the dam to collect any water picked up in the downstream shell or in the gravel-sand blanket.

<u>Spillway Section:</u>

The concrete overflow (ogee) spillway section of the dam proposed under this alternate will be located on the southern side of the Thames River Valley as this location generally places the spillway in line with the existing Highway 19 and the Canadian National Railway bridges crossing the South Branch of the Thames River immediately downstream from the dam. This location is also approximately at the centre of mass of flow. The Thames River Valley is gently sloping laterally in this area to the dry weather flow channel located on the south side of the valley and then rises sharply up toward the City of Woodstock. The south training wall has been kept a distance of approximately 200 feet from this sharply rising side of the valley to prevent scour of the valley wall by discharge over the spillway.

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(1) "Design of Small Dams," United States Dept. of the Interior, Bureau of Reclamation, U. S. Government Printing Office, Washington, D. C., 1960, pgs. 202-204.

The proposed concrete overflow (ogee) spillway section will have a total length of 137 feet between the training walls with an overflow crest length of 105 feet on which will be mounted five 21 feet wide by 24 feet high radial (Tainter) type gates. Four 8 feet wide piers are also located on the ogee spillway section which divide the overflow crest length into the five equal 21 feet wide segments or bays. These piers take the thrust of the radial gates and provide support for the machinery deck.

The crest of this proposed concrete overflow spillway section was set at Elevation 927.5, 7.5 feet above the probable bottom of the reservoir in this area. This selected elevation was a compromise between the desire to provide the maximum gate depth possible so that most of the storage volume in the reservoir would be useable and not wasted as dead storage below the crest of the dam and the desire to keep the bottom of the gates above any silt that will be deposited behind the dam, which could conceivably interfere with the operation of the gates. Setting the crest of the spillway at this elevation will result in a permanent pool (dead storage area) of 651 acre-feet as previously mentioned.

Both the profile of the upstream and downstream faces of the concrete overflow spillway section were designed in accordance with data based on hydraulic laboratory tests of the United States Bureau of Reclamation. The face of the spillway crest downstream from the axis of the dam is defined by the equation

$$Y = 13.1\left[\frac{X}{25}\right]^{1.748}$$

The face of the spillway crest upstream from the axis of the dam is defined by a single curve with a radius of 11.50 feet with a point of tangency at a distance of 4.50 feet from the axis of the dam. The back face of the overflow section below this point of tangency was placed on a slope of 1 : 1 so as to provide stability of the spillway section and to reduce the coefficient of discharge over the spillway.

The five 21 feet x 24 feet radial (Tainter) type gates are positioned on the spillway crest so that the gate sill intersects the face of the spillway downstream and approximately one foot below the crest of the spillway so as to prevent cavitation of the face of the spillway at partial gate openings. A freeboard of 0.5 feet above the

maximum water surface elevation of 950.0 (full pool elevation) has been provided to prevent flow over the top of the gates. The gate trunions have been located just above the water surface of the maximum probable flood nappe to avoid contact with floating ice and debris and have been placed below half the depth of the gate above the sill, in order to transmit the maximum reaction as horizontally as possible to the trunion girder. Each radial gate is to be operated by a separate hoist and motor located on the machinery deck, so that the gates can be operated singularly or at the same time. The gates will be operated by automatic controls from an operations house, which will be located off the dam in the vicinity of the parking lot and access road as previously mentioned. Heating equipment will be provided to keep the gates operable during freezing weather.

During the preparation of this report, consideration was given to the placing of three 35.5 feet wide by 24.0 feet high radial gates on the crest of the concrete overflow (ogee) spillway section with two supporting piers rather than the five gates tentatively adopted and thereby save the cost of the additional piers and with a possible reduction in the gate costs due to the larger sizes. However, the larger size gates were not adopted as the additional costs of the machinery deck caused by the greater spacing between piers and the heavier hoisting equipment required generally nullified any cost saving incurred and greatly reduced flexibility of operation. Consideration was also given to using a vertical lift type of gate instead of the radial type gages. However, the many apparent advantages of the radial gate over the vertical lift gate such as lighter hoisting machinery, quicker lifting time, elimination of vertical lift gate slots with their attendant cavitation and generally cheaper cost, made the choice of the radial type of gate mandatory for this report. Further consideration of gate type and size will be made during the preparation of the final plans, if Alternate 1 is adopted.

Stop log slots have been provided in the piers and training walls, as shown on Drawing No. 6, so that each gate bay between the piers can be closed in an emergency by the insertion of stop logs. Steel stop logs will be stockpiled off the dam site. No special device will be provided for the placing of these stop logs as it is expected that they will be rarely used in which case a small truck mounted crane can be employed.

A 36 inch diameter outlet tube to discharge small rates of flow such as would be required for discharge from the low flow maintenance pool has been provided in the south training wall of the concrete spillway (ogee) section. A 36 inch motor operated butterfly valve to open and close the tube will be operated from a dry type outlet control chamber adjacent to the south seep wall. The elevation and details of the outlet tube will be determined during the preparation of the final plans.

The 105 foot wide overflow (ogee) spillway crest will be capable of passing 45,000 cfs., the peak discharge of the maximum probable flood (spillway hydrograph) modified by the storage in the reservoir, at a maximum reservoir water surface elevation of 950.0, with the gates in the full open position, at a head of 22.5 feet on the crest of the spillway. The maximum probable flood inflow hydrograph (peak = 55,000 cfs) and the maximum probable flood outflow hydrograph modified by the storage in the reservoir (peak = 45,000 cfs) as determined by flood routing through the reservoir storage is shown on Figure 1.

Stilling Basin:

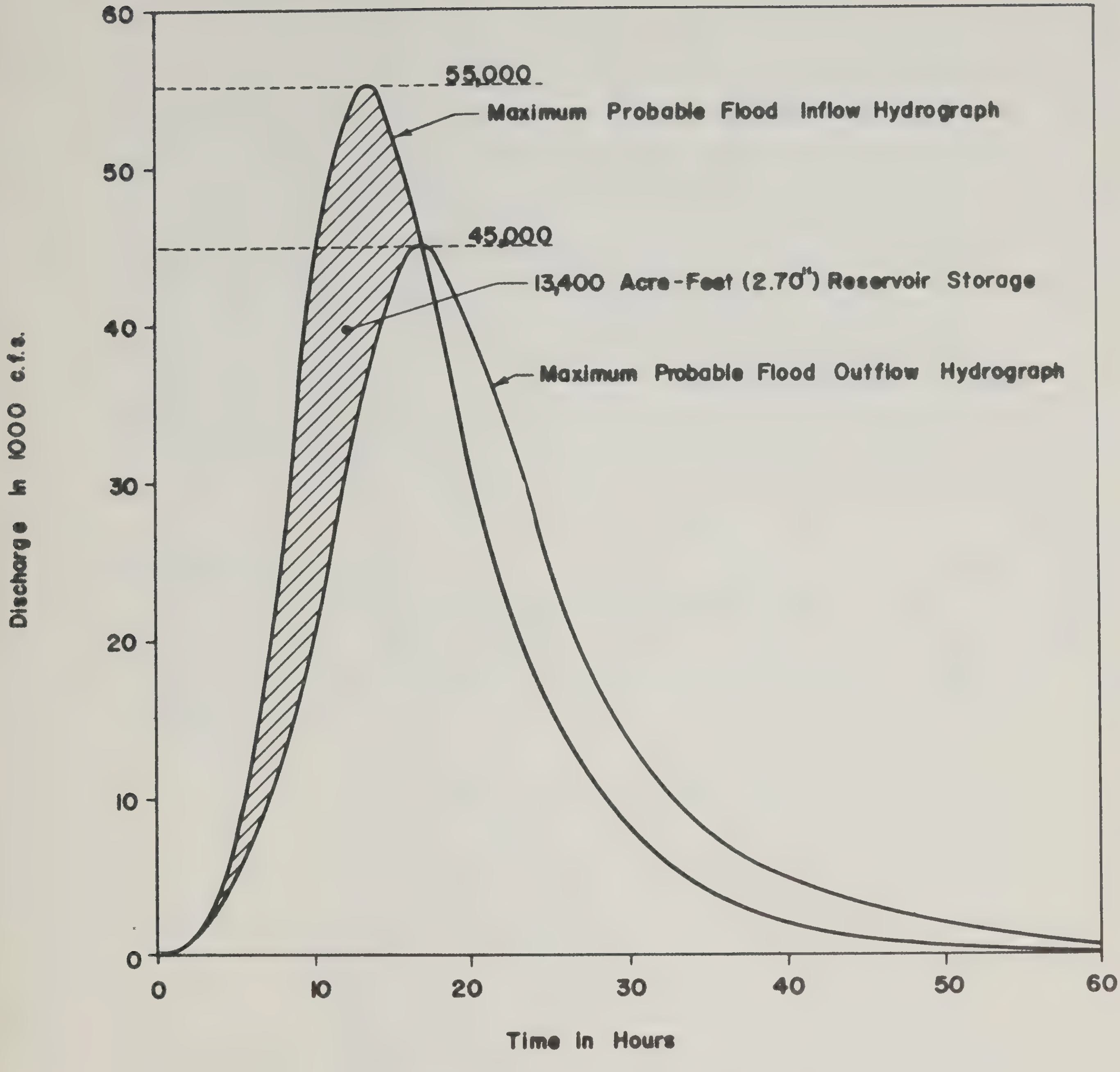
In order to design a stilling basin for the spillway, the downstream depth of flow (d_1) at the toe of the spillway, the velocity of flow (v_1) at the toe, the Froude number (F_1), the conjugate depth (d_2) and the length of the hydraulic jump (L) with no stilling basin were computed at various discharges (Q) and are tabulated in Table 5 below.

TABLE 5

Stilling Basin Characteristics - Dam on South Branch of Thames River

Alternate 1 - High Dam

Q (cfs)	d_1 (ft.)	v_1 (ft/sec.)	F_1	d_2 (ft.)	L (ft.)
8,000	11.72	44.3	5.94	7.9	83
16,000	3.55	43.0	4.02	5.1	105
30,000	7.10	40.2	2.66	3.2	113
45,000	11.81	36.3	1.86	2.1	100



MAXIMUM PROBABLE FLOOD
(SPILLWAY INFLOW & OUTFLOW HYDROGRAPHS)
SOUTH BRANCH THAMES RIVER
WOODSTOCK DAM
ALTERNATE I — HIGH DAM

FIGURE I

A graph showing the relationship of the conjugate depth curve and tailwater rating curve is shown on Figure 2.

Due to the low Froude numbers (F_1) of 2.0 to 6.0 at the high discharges caused by the low head on the spillway and the resultant low velocity of flow at the spillway toe, an oscillating type of hydraulic jump will occur. The design of an effective stilling basin is difficult, due to the oscillating position of the flow jet on the apron and the resulting waves and rough surface condition created, and should generally be determined by model tests. For purposes of this report, a flat stilling basin apron, with a length equal to the average length of the hydraulic jump throughout the range of discharges and without any chute piers or baffle blocks other than an end sill, was considered a reasonable design for the conditions involved and is shown on the preliminary drawings. Upon selection of either Alternate 1 or Alternate 2, model tests will probably be made to determine the best length of the basin and the desirability of any chute piers or baffles.

Roadway Relocations:

In conjunction with the construction of a dam at this site and the required railroad location, certain roadways will have to be relocated and/or raised. Cost estimates for these roadway alterations are included in the items listed in the section of this report on Cost Estimates.

1. The existing Innerkip Road (County Road 4) will be flooded out by the proposed maximum pool elevation (Elevation 950.0). As this road is an important traffic artery, it will be realigned and raised above the water surface to a minimum elevation of 956.0 as shown on Drawing No. 11. In conjunction with the realignment of this road a new 225 foot span bridge (3 @ 75 feet) will be constructed over the South Branch of the Thames River. The details of the bridge are shown on Drawing No. 7. The existing Innerkip Road bridge will be demolished or salvaged and the existing roadway embankment removed.

2. The existing side road between Lots 5 and 6, in Concession 15 of East Zorra Township, (near

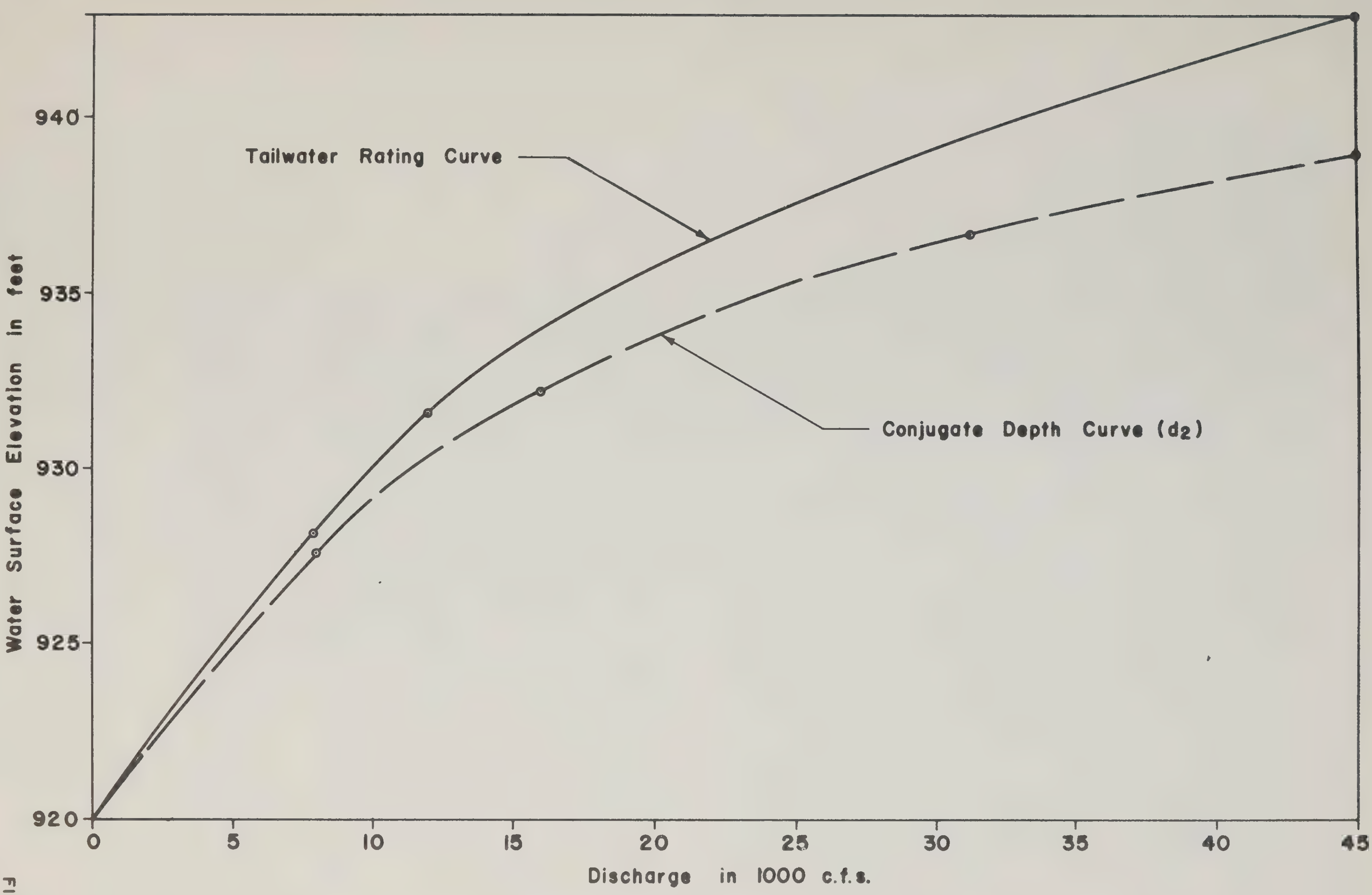
Innerkip Road) will have to be realigned and raised as shown on Drawing No. 11. A new 30 foot span, reinforced concrete bridge will be required to carry this relocated road over Timms' Creek. As this is an extremely simple type of bridge, no detailed drawing of it has been included in this report. The existing reinforced concrete bridge on the old roadway alignment will be demolished.

3. The existing Huron Street crossing of the South Branch of the Thames River, near the City of Woodstock, will be dead ended. Huron Street on the north side of the Thames River Valley will be diverted in a westerly direction and run parallel to the proposed relocated Canadian Pacific Railway tracks to Highway 19, by way of Fredrick Street. Huron Street, on the south side of the valley will be turned directly into Highland Drive, also shown on Drawing No. 8. The existing Huron Street bridge across the South Branch of the Thames River and its approach fills will be removed.

4. The grade along a short portion of the existing Innerkip Road (County Road 4) crossing Lampman's Drain, a minor tributary of the South Branch of the Thames River, will have to be raised and a new 15 foot single span bridge or culvert will be constructed at this location.

5. The grade along a short portion of the existing Given Road between Lots 15 and 16, in Concession 2, in the Township of Blandford, adjacent to the existing Canadian Pacific Railway tracks, will have to be raised.

6. The existing Given Road between Lots 15 and 16 in Concession 2, in the Township of Blandford will be raised for a short distance over Lampman's Drain, and a new 15 foot single span bridge or culvert will be constructed at this location.



TAILWATER RATING & CONJUGATE DEPTH CURVES
WOODSTOCK DAM
ALTERNATE I - HIGH DAM

FIGURE 2

Utility Relocation:

In conjunction with the construction of the dam on the South Branch of the Thames River, as proposed under this alternate, with a resultant full pool level elevation of 950.0, certain utilities such as sewers, pipe lines and hydro lines, will have to be relocated. A cost estimate for these necessary utility relocations, is given in the section of this report on Cost Estimates.

1. An 8 inch diameter petroleum products pipeline, owned by the Sun Canadian Pipe Line Co. Ltd., crossing the South Branch of the Thames River at mile 23.0, will be relocated around the proposed reservoir area. Due to this relocation, the length of this pipe will increase from 4,200 feet to 7,300 feet between end points. Alternate solutions to this relocation were to weigh down the existing pipe to prevent flotation or to place a new and heavier wall pipe (river crossing pipe) at approximately the same location as the existing pipe. However, due to the dangers involved, the potential difficulties of pipe maintenance and the small cost differential between the various alternates, the relocation plan was adopted.

2. A 12 inch diameter petroleum products pipeline owned by the Sarnia Products Pipeline Division of the Imperial Oil Company, Ltd., crossing the South Branch of the Thames River at Mile 21.7, must be replaced by a heavier wall river crossing type pipe at the same location as shown on Drawing No. 12. Also in the proposed shallow flooded areas east of the Thames River, the existing pipeline although not being replaced, must be weighted down to prevent flotation. The alternate solution of relocating the pipeline around the reservoir area, although more desirable from the maintenance viewpoint, was prohibitively expensive and was not considered economically feasible.

3. An existing 27 inch diameter combined sewer owned by the City of Woodstock and running west and adjacent to the existing Canadian Pacific Railway tracks

on the south side of the South Branch of the Thames River valley, from 1,150 feet east of Huron Street to 2,000 feet each of Highway 19, will be flooded out by the proposed water surface of the reservoir and will have to be relocated in a southerly direction. At the present time, this existing sewer has an overflow chamber at Huron Street, which, when the sewer becomes overloaded, discharges flow (including raw sewage) directly into the South Branch of the Thames River. As this overflow chamber is upstream from the proposed dam site, it must be moved downstream from the dam to prevent discharge of raw sewage into the reservoir. In order to prevent surcharge and backing up of the existing sewer above the relocated section, a larger pipe than now exists (probably 60 inch diameter) will have to be installed, to carry the maximum load down to the new overflow chamber. It will also be necessary to place a new discharge pipe from the proposed overflow chamber to the river. This proposed sewer will be relocated so as to run under Highland Drive and the proposed Rivercrest Drive access road to the dam. The joints of this proposed pipe will be of the rubber gasket type so as to prevent possible infiltration into the sewer due to the head that will be placed on the sewer by the high water surface elevation of the reservoir and a possible increase in load on the Woodstock Sewage Treatment Plant. In the vicinity of the dam the proposed 60 inch diameter pipe will be fitted with anti-seep collars and the trench backfilled with impervious material.

The existing 27 inch diameter sewer in the vicinity of the dam site will be completely removed when excavating for the impervious cutoff trench. The remainder of the line will be blocked off at each manhole by a concrete plug and the manhole chimneys will be filled up.

4. Seven existing telephone poles and 2,300 feet of aerial cable owned by the Bell Telephone Company of Canada, crossing the Thames River Valley at

Huron Street, will be replaced by 2,300 feet of buried 100 pair submarine type cable since Huron Street will be dead ended and the roadway fill and bridge removed.

5. Two existing telephone poles and 100 feet of aerial cable, owned by the Bell Telephone Company of Canada and located at the proposed new grade crossing of the relocated Canadian Pacific Railway tracks at Huron Street on the north side of the Thames River Valley, will have to be replaced by 100 feet of buried 100 pair cable.

6. Two 45 foot railway crossing poles, owned by the Bell Telephone Company of Canada and located at the existing Highway 19 grade crossing of the Canadian Pacific Railway, will have to be removed and two new 45 foot railway crossing poles placed at the new grade crossing.

7. Several telephone poles and cables, owned by the Oxford Telephone Company and crossing the South Branch of the Thames River in the vicinity of Innerkip Road (County Road 4), must be relocated to the proposed shoulder of relocated Innerkip Road.

8. Certain power lines and poles owned by Ontario Hydro located at various places throughout the reservoir area must also be relocated.

RELOCATION OF THE CANADIAN PACIFIC RAILWAY

Under Alternate 1, the proposed maximum water surface of the reservoir (Elevation 950.0) will flood out the existing tracks of the Canadian Pacific Railway, between the site of the dam at Woodstock and Innerkip, thus requiring a relocation of the tracks in this area.

From a general study of the area involved, it was immediately apparent that the railroad should be relocated from the south side of the Thames River Valley to the north side of the valley, for the following reasons:

1. Much less interruption or interference with the operation of the railroad would be necessary during the construction period as compared to either keeping the tracks in the same location and raising them, or by the construction of new tracks adjacent to the old, but at a higher elevation.

2. The relocation of the tracks to the north side of the valley provided a greater distance in which to rise up to meet Elevation 956.0, the minimum permissable elevation of the tracks adjacent to the reservoir (the maximum reservoir water surface elevation of 950.0 + 6.0 ft. of freeboard to the top of the rails) at the maximum permissable grade of 0.4%. If the tracks were to remain on the south side of the valley it would be necessary to shift the centerline of the dam a considerable distance to the east, to conform with the maximum permissable grade requirements, thus losing valuable reservoir storage.

3. Easier access to the reservoir from the City of Woodstock would be made possible by this shifting of the tracks from the south side to the north side of the valley.

4. A better horizontal and vertical alignment as well as a better earthwork balance could be made in the larger more open area on the north side of the valley.

5. Less interference with county roads would result.

6. The land adjacent to the south side of the existing tracks, in the vicinity of Woodstock, which would probably be occupied by the relocated tracks if located on the south side of the valley, was needed for the relocation of an existing sewer now running parallel to the existing tracks in this area.

The horizontal alignment of the relocated Canadian Pacific Railway tracks, shown on Drawings Nos. 8 through 12 inclusive, and the vertical alignment of the tracks, shown on Drawings Nos. 13 through 15 inclusive, were made in accordance with design criteria specified by the Canadian Pacific Railway. A typical cross section of the proposed railroad relocation is shown on Drawing No. 8. The alignment of the railway relocation, as shown on these drawings, was selected so as to generally keep the railroad out of long cuts without a great departure from the desirable situation of balanced cuts and fills resulting in much of the railroad relocation being along the wall of the valley in what is classified as a side hill cut situation. It was anticipated in the setting of the vertical alignment that material excavated from the dam site sould immediately be rolled into place in the railroad embankment in front of the dam.

The only major horizontal curve on the proposed relocation of the railroad occurs in the vicinity of the dam where it was necessary to move the tracks around the north abutment of the dam with a 3 degree curve, the maximum permissible degree of curvature. This requirement actually set the position of the dam since it was deemed desirable to keep the end of the dam a minimum of at least 200 feet from the tracks, so as to prevent vibration caused by passing trains from having any adverse affect on the rolled earth embankment. A smaller degree of curvature would have required that the dam be moved upstream (east) thus losing valuable storage. To conform with the vertical alignment design criteria, the grade in the region of this 3 degree curve was reduced from 0.4% to 0.25%. The number of grade crossings of the relocated Canadian Pacific Railway tracks will remain the same as at the present location.

The proposed grade crossings will be at the following locations:

1. At Highway 19, near the South Branch of the Thames River, just north of the old grade crossing, as shown on Drawing No. 8. A slight rise in the elevation of the roadbed of Highway 19 will be necessary at this location to meet the new grade of the railroad.

2. At Innerkip Road near the South Branch of the Thames River as shown on Drawing No. 11. A change in both the horizontal and vertical alignments of Innerkip Road and the Concession Road in this area is also being made.

3. At Huron Street on the north side of the valley as shown on Drawing No. 9. To provide access to the land adjacent to the reservoir, Huron Street will have to be lowered approximately 10 feet, in this area to meet the railroad grade at this point. Huron Street itself will be diverted in a westerly direction and run parallel to the proposed railroad tracks toward Highway 19.

Access to the dam itself will be from the south side of the Thames River Valley from Rivercrest Drive, and no access to the dam or reservoir area from the north side of the valley across the tracks between Highway 19 and Huron Street, will be necessary or is contemplated other than at Huron Street. All existing farm roads will be dead ended just north of the railroad by means of a barrier fence and the railroad right-of-way will be completely enclosed by a suitable type fence to prevent accidents.

Bridges:

Two bridges will be required to carry the relocated Canadian Pacific Railway over the South Branch of the Thames River and its tributaries.

1. At 284 feet, four span (4 @ 71 feet) through plate girder type bridge carrying the relocated Canadian Pacific Railway over the South Branch of the Thames River at Station 10 + 00, 2,000 feet downstream from the centreline of the dam and just upstream from the existing Highway 19 bridge. The details of this proposed bridge as shown on Drawing No. 16. The maximum allowable railroad grade of 0.4% from the existing fixed elevation of the

freight marshaling yard in Woodstock along with the high tail water elevation of the maximum probable design flood (45,000 cfs) dictated the use of this type of plate girder design rather than the more desirable deck plate girder type of structure. The lower elevation of the steel of a deck plate girder structure in this situation with the same railroad grade would have resulted in an orifice condition through the bridge with a large increase in head loss through the structure. This would have reduced the coefficient of discharge over the spillway due to submergence and thus would have increased the length and cost of the spillway. Also the horizontal thrust of such submergence would have increased the cost of a deck plate girder type of structure at this location. More rip rap would be required for slope protection on the downstream face of the dam.

In order to keep the skew angle of the bridge down to 45° and thus keep the cost of the structure down, a slight curve was placed in the realigned channel at this point.

It is proposed to place the north abutment of this structure on piles since it will be situated on freshly rolled earth fill. The foundations of the south abutment will be placed upon and be supported by existing ground. The upstream, east side of the railroad embankment in this area will be protected from erosion by heavy rip rap slope protection.

2. A 30 foot single span rolled beam bridge carrying the relocated Canadian Pacific Railway over Timm's Creek, at Station 208 + 00. Since this is a simple type of bridge, no detailed drawing of it has been prepared for this report. The south side of the proposed railroad embankment in this area will be protected by heavy rip rap slope protection.

CEDAR CREEK CHANNEL IMPROVEMENT

The Cedar Creek drainage basin, like most of the other sub-drainage basins in the watershed of the South Branch of the Thames River, consists of fairly flat gently rolling heavily farmed lands. The highest point in the 33.2 square mile drainage basin is only about 140 feet higher than the point of its intersection with the South Branch of the Thames River near Woodstock. The actual size of the dry weather channel is quite small which when combined with the very flat slope of the stream results in a limited capacity to transmit flow; most of the needed capacity in flood times being in the overbank region.

The highly developed system of municipal drains in that portion of the drainage basin above the Highway 401 bridges makes Cedar Creek much more of a "flashy stream" than might normally be expected. Minor floods causing some damage have occurred. Most of the floods have been the result of snowmelt in the spring as is the situation of the rest of the Thames River drainage basin.

The creek itself, for purposes of discussion, can be divided up into two parts: the upper portion of the creek draining the rural farm land above Parkinson Road; and the lower portion of the creek passing through and draining a portion of the urbanized area of the City of Woodstock.

Generally, the people engaged in farming the upper rural areas adjacent to Cedar Creek have recognized, through observation or hard experience, the necessity of keeping the overbank region clear and open and normally keep this flood plain land as permanent meadow or pasture land, so that little or no damage occurs during periodic flooding of the creek in this region.

The lower portion of the channel draining the urbanized area of Woodstock can be further subdivided into two sections as follows:

1. That section of the drainage basin from Mill Street Bridge to Parkinson Road Bridge, where the channel runs through Southside Park, and the general open area between the Henry Street Bridge and Mill Street Bridge. Little or no flood damage has occurred in this section.

2. That section of the Cedar Creek drainage basin from Mill Street Bridge to the South Branch of the Thames River, the flood plain of which is highly built up with small factories and buildings and where much flood damage has occured in the past. Some of the flood damage in this region has occured due to the flooding by backwater from the main stem of the South Branch of the Thames River as the land adjacent to Cedar Creek in this area is also in the flood plain of the South Branch of the Thames River. Food damage from this source will be eliminated by the Woodstock Channel Improvement of the South Branch of the Thames River, which is described in another section of the report.

In order to ascertain which areas are subject to flooding, under conditions as they now exist, and the probable causes of such flooding (e. g. inadequate bridge waterway areas), a series of water surface profiles were computed along the length of the creek from the South Branch of the Thames River to the proposed dam site on Cedar Creek for discharges ranging from 2,000 cfs to 8,000 cfs, the maximum peak discharge of the design flood hydrograph as can be seen from Figure 11, titled "Design Flood Hydrograph, Cedar Creek, at Entrance South Branch of Thames River, (Mile 0.0).

In the computation of these water surface curves, the following assumptions were made:

1. The starting water surface elevation (concurrent water surface elevation in the South Branch of the Thames River) was assumed to be Elevation 921.5.

2. The existing abandoned Canadian Pacific Railway bridge over Cedar Creek adjacent to the new bowling alley at the intersection of Main and Mill Streets, which constitutes a severe channel construction had been removed.

The methods and other general assumptions used to compute these water surface profiles are outlined in detail in Appendix B of this report.

The computed water surface profiles for the existing conditions are shown on Drawings Nos. 36 through 38. The areas that will be covered by flood water at each of the above discharges are shown on Drawings Nos. 17 and 18.

From Drawing No. 18, it is quite apparent that very little damage could be done in the region between Mill Street and Parkinson Road as the flood plain in this region is relatively open and unoccupied at the present time, as previously mentioned, such that no channel improvement or local flood protection project in this area appears warranted at this time. The existing Butler and Finkle Street Bridges and their approaches are quite low so that major floods will pass over them without much head loss involved and extensive resultant upstream flooding. The minor damage that would be suffered by the Ontario Hydro Transformer Station, the several unused buildings of the Overland Express Company located on the north side of Southside Park, and the other few houses located on this area, will be considerably reduced by the proposed channel improvement downstream from Mill Street, which will be discussed in subsequent paragraphs. The Transformer Station and the Overland Express Company buildings can be further protected by local flood protection measures such as flood walls, levees and building flood proofing.

However, every effort should be made to keep the area between Mill Street and Finkle Street as an open unoccupied area so that future flood damages to any new building or structure in the area would not occur. This may be done either by the process of flood plain zoning or by outright acquisition of the flood plain property in this region, much of which is still in private hands. Suggested limits or boundaries of flood plain zoning are shown on Drawing No. 19, which also shows the areas which will be covered by flood water with the assumed downstream channel improvement in place.

From Drawing No. 17, it is quite apparent that most of the flood damage will occur in the small, heavily built-up area adjacent to the Cedar Creek Channel between the South Branch of the Thames River and Mill Street, as previously mentioned. This statement has been borne out by past experience.

From the computed water surface profiles for the existing conditions along Cedar Creek (Drawing No. 36), it is also apparent that the flooding in this area is aggravated by inadequate bridges such as

the existing Canadian Pacific Railway timber trestle bridge over Cedar Creek, just downstream from the Ingersoll Road bridge, whose closely spaced trestle bents result in severe flow constriction and artificial man-made constrictions and bends which compound the basic problem of flood plain occupancy and a small dry weather flow channel.

Based on the values of the properties in this area subject to flood damage, a local flood control project in the form of a channel improvement is definitely warranted in this area.

After a thorough study of the problems involved, a trapezoidal channel with a 75 foot bottom width and 2:1 side slopes was selected as the size of the proposed channel improvement in this area. The proposed alignment and cross-sections are shown on Drawing No. 20. At the proposed channel bottom slope of 0.153%, this channel will carry 6,000 cfs at a depth of 8 feet; the approximate bank-full stage in this region and the 8,200 cfs Design Flood with only minor flooding and very little damage.

Theoretically, in order to provide full protection against flooding in this area, the proposed Cedar Creek Channel should be large enough to carry the peak flow (8,200 cfs) of the Design Flood Hydrograph within the channel banks, perhaps even with some free-board.

However, in the case of Cedar Creek, as is the case in many local flood protection projects, the benefits which would be derived from providing a sufficiently large channel to fully contain this maximum peak are not considered to be warranted due to the following reasons:

1. Although much flood damage has occured in this area, it has not been extremely serious.

2. The frequency of the design flood is quite high (100 years).

3. A channel sufficiently large to contain fully the design flood would cause considerable damage to adjacent property, which, when combined with the extra cost of the wider channel improvement and longer bridges involved would be prohibitively expensive.

In conjunction with this proposed channel improvement between the South Branch of the Thames River and Mill Street, certain additional bridge work and other improvements will be required as follows:

1. A new steel girder bridge will be constructed to carry the Canadian Pacific Railway (St. Thomas sub-division) over Cedar Creek to replace the existing timber trestle bridge. The details of this proposed bridge are shown on Drawing No. 21.

2. An additional span must be added to the Ingersoll Road Bridge over Cedar Creek. A preliminary plan of the work involved in shown on Drawing No. 22.

3. A new footbridge over Cedar Creek in the Park will be required to replace the existing footbridge due to the Channel widening involved.

4. A new bridge will be required to carry Main Street over Cedar Creek as a result of the new channel alignment in this area. In order to get sufficient height to pass the flow of 6,000 cfs under the bridge, the grade of Main Street will have to be raised 3 to 4 feet in this area. Demolition of the existing Main Street Bridge will also be required. A preliminary plan of this proposed new bridge is shown on Drawing No. 23.

5. The existing abandoned Canadian Pacific Railway Bridge over Cedar Creek adjacent to the new bowling alley will be removed.

6. An existing abandoned factory building, on the side of Cedar Creek adjacent to Ingersoll Road, will have to be demolished.

7. Certain alterations to the loading platform behind the Cold Storage Warehouse will have to be made to reduce the sharp curve in the existing channel.

A cost estimate of the total work required for the Cedar Creek Channel Improvement is given in the section of this report on Cost Estimates.

Utility Relocations:

In conjunction with the proposed construction of the Cedar Creek Channel Improvement, certain water mains and sanitary sewer siphons would have to be moved or re-located. In most cases, it will be necessary to keep the water mains or sanitary sewer siphons in operation except perhaps for short periods of time at night. Costs estimates were therefore made on this basis, utilizing the following general sequence of constructions:

1. Construct the existing water main or sanitary sewer siphon in a trench adjacent to the existing utility, making all pressure tests and sterilization and back-filling as may be required without actually tapping in or making any connections at the ends.

2. During periods of low flow or demand (at night or early in the morning) make a rapid connection between the ends of the new line and the old line.

3. Abandon or remove the portion or section of the old line no longer in use.

The water mains and sanitary sewer siphons which cross Cedar Creek, all of which will be affected by the Channel Improvement, are listed below. The stationing is that of the proposed channel improvement shown on Drawing No. 20.

1. Sanitary sewer siphon (2-14 inch cast iron pipes) 600 feet downstream from existing Canadian Pacific Railway Bridge (Sta. 12+00±).

2. Sanitary sewer siphon (2-24 inch cast iron pipes) between Ingersoll Road and the Canadian Pacific Railway Bridges (Sta. 21+00±).

3. Water main (8 inch cast iron pipe) at Ingersoll Road (Sta. 22+00±).

4. Sanitary sewer siphon (8 inch cast iron pipe) in the vicinity of the Cold Storage Warehouse (Sta. 31+00±).

5. Water main (unknown size and type) in the vicinity of the Cold Storage Warehouse (Sta. 32+00$\pm$).

6. Water main (unknown size and type) in the vicinity of the Cold Storage Warehouse (Sta. 34+00$\pm$).

Certain minor re-location of poles and lines of the Bell Telephone Company of Canada and the Woodstock Public Utilities Commission will also be required.

SECTION 6 - COMPONENTS AND ENGINEERING FEATURES OF ALTERNATE 2

DESCRIPTION

Under this alternate, two small low level rolled earth fill dams with gated concrete spillway sections will be constructed in the drainage basin of the South Branch of the Thames River as previously mentioned. One dam site is on the South Branch of the Thames River near the City of Woodstock, Ontario. The details of this proposed structure are shown on Drawing Nos. 24 and 25. The other dam site is on Cedar Creek, a major tributary of the South Branch of the Thames River, also near the City of Woodstock, Ontario. The details of this proposed structure are shown on Drawing Nos. 26 and 27. The locations of these proposed reservoirs are shown on Drawing No. 1.

LOW LEVEL DAM - SOUTH BRANCH OF THAMES RIVER

General:

Under this alternate, the proposed dam will be located 1,800 feet upstream (east) of Highway 19, somewhat downstream from the dam site as proposed under Alternate 1. This location is as far downstream as possible in order to provide the maximum amount of reservoir storage within the height limitations imposed on the maximum pool elevation by the adjacent Canadian Pacific Railway to remain in place. The maximum water surface elevation of the reservoir was set at Elevation 940.0, the elevation set in the original Upper Thames Valley Report.

With the water surface at this elevation, the useable water storage volume created by this dam will be 5,046 acre-feet. An additional 740 acre-feet will exist as permanent dead storage below the crest of the dam. When the water surface is at this maximum elevation of 940.0, a land area of 640 acres will be flooded.

Storage-elevation and area-elevation curves for this proposed reservoir are shown on Drawing No. 25. A topographic map showing the location of the proposed dam site and the reservior area, is shown on Drawing No. 2.

Access to the proposed dam and machinery deck will be from relocated Huron Street on the north side of the Thames River Valley to avoid a grade crossing of the existing Canadian Pacific Railway tracks. A parking lot will be constructed in this area to provide necessary parking space for employees and visitors. Both the proposed road and parking lot are shown on Drawing No. 24. Under this Alternate, a small operations house and administration building will be constructed on the north side of the valley in the vicinity of the proposed parking lot.

Soils and Foundations Conditions:

In order to determine foundation conditions, a preliminary soils exploration and rock drilling program along with hydraulic pressure testing of the underlying rock was conducted at this proposed

dam site. The soils exploration and drilling program consisted of placing 8 cased hole borings through the soil overburden to rock and then coring the bedrock for depths of 30 feet. This was followed by hydraulic pressure testing of the rock in certain of the holes to locate seams which would transmit pressure as was done for the high dam of Alternate 1. In addition, two permanent observation wells were drilled along the same line of cased holes for the Ontario Water Resources Commission, for purposes of observing fluctuations of the ground water table. Both yielded information as to the type of soil and rock. The soils and rock profile along the centreline of this proposed dam, as determined by the soils exploration and rock drilling program, is shown on Drawing No. 4. The soils and foundation conditions encountered and the proposed foundation treatment and methods are discussed in the following paragraphs. As previously mentioned in this report, all soils were classified in accordance with the United Soil Classification System.

The soils at this dam site were found to consist of glacial till and alluvium deposits, overlying limestone at a shallow depth. The limestone bedrock is encountered at depths of 14 feet in the stream flood plain to 51 feet at the valley walls.

The actual soils and bedrock conditions at this dam site were just about identical to the soils and bedrock conditions at the alternate upstream dam site.

Generally, the soil exploration and rock drilling program undertaken at this site showed that the foundation conditions for a dam at this site are also fairly good except for seepage which will be discussed in subsequent paragraphs. Bedrock was found to be approximately at Elevation 906.0 at the centre of the valley, which, although would provide excellent support for the dam, was deeper than at the upstream dam site, thus requiring a deeper foundation and more concrete.

If a dam is constructed at this site, it will be necessary to remove or strip the soft weathered top layer of rock in this area.

As the rock underlying this dam site is extensively fractured, it will be necessary to grout the bedrock at this site for an average depth of 33 feet under the full length of both the spillway and embankment sections. The grouting will be done

through a concrete grout cap 8 feet wide by 3 feet deep underneath the rolled embankment sections and through the first pour of the concrete overflow spillway (ogee) section, (probably 4 feet in depth) as previously described for the Alternate 1 upstream dam.

Generally, both the glacial till and allevial type soils, comprising the overburden above the bedrock at this site, will be satisfactory as foundation material for the earth embankment and no special treatment for settlement or stability will be required. However, to prevent seepage through this region, an impervious cutoff trench to bedrock, under the full length of the earth embankments on each side of the spillway, will be required.

As was the case in the upstream dam site of Alternate 1, the bottom width of this impervious cutoff trench was set at 20 feet. Dewatering of the dam site at this area will also be necessary for placement of the trench.

The borings also indicated two other possible areas of seepage loss at this dam site:

1. Appreciable seepage losses can be expected through the extensive alluvial river terrace on the south side of the Thames River Valley beyond the abutment of the dam. Since the lateral limit of the cutoff trench is limited by the existing Canadian Pacific Railway tracks in this area, it will be necessary to place a clay blanket over this deposit to a point 300 feet upstream, to reduce seepage losses through the region shown as Section CC of the Embankment Section on Drawing No. 24.

2. Seepage losses are also possible through the bedrock beyond the grout curtain, similar to the upstream situation at the Alternate 1 upstream location.

Embankment Section:

The proposed rolled earth sections of the dam are to be of the zoned embankment type, with an impervious central core wall to prevent seepage, surrounded by a shell of free draining material similar to that of Alternate 1. The embankment sections

for this proposed dam are shown on Drawing No. 24. Satisfactory material for the impervious core is available as sandy-clay (SC) and gravel-sand-clay (GC) in adjacent glacial till deposits, and free draining sand and gravel for the shell is available in adjacent sand pits in the form of well graded sands (SW) or gravelly sands (GP). As mentioned under Alternate 1, there is also the possibility that some of the silty-sand material (SM or SP), excavated from the cutoff trench and the spillway, could be satisfactorily used for the shell material. However, because of the great variability of this material and the large percentage of fines contained by it, the decision was made to waste this excavated material and bring in new, better graded, shell material from the adjacent sand pits, for purposes of this report.

After much study of the shell material mixture that would be used in the construction of the dam, a slope of 2 −1/4 : 1 was also selected as the slope of both the upstream and downstream faces of this dam for purposes of this report.

The selected dimensions of the impervious core are the same as for the dam site proposed under Alternate 1.

The proposed embankment will have a crest width of 18 feet and a freeboard of 6 feet (top of the embankment at Elevation 946.0) in conformance with the recommendations and formula of the United States Bureau of Reclamation. Both the upstream and downstream faces of this proposed structure would be protected by rip rap. The rip rap on the upstream face will extend to the top of the dam as a protection against wave erosion. The rip rap on the downstream face will extend from the ground to Elevation 935.0, to protect the embankment from possible erosion due to the extremely high tailwater condition. The remainder of the downstream face will be sodded. A 1-foot thick gravel-sand layer or blanket (GW classification) will be provided under the rip rap slope protection to prevent fines from being washed out through the rip rap by wave action and to ensure that the phreatic line stays within the toe of the embankment, similar to the arrangement for Alternate 1. A toe drain will be installed along the downstream toe of the dam to collect any water picked up in the downstream shell or in the gravel-sand blanket.

Spillway Section:

The concrete overflow (ogee) spillway section of the proposed dam at this site was placed as close to the southern side of the Thames River Valley as possible so as to generally place the spillway in line with the existing Highway 19 and Canadian National Railway Bridge, crossing the South Branch of the Thames River immediately downstream from the dam. This location is also approximately at the centre of the mass of flow. In order to prevent possible erosion and scour of the previously mentioned alluvial river terrace on the south side of the Thames River Valley due to discharge over the spillway, a rip rap facing has been provided in this area as shown on Embankment Section CC on Drawing No. 24.

The proposed concrete overflow (ogee) spillway section will have a total length of 659 feet between the training walls, with an overflow crest length of 483 feet, on which will be mounted twenty-three 21 feet wide by 14 feet high radial (Tainter) type gates. Twenty-two 8 feet wide piers are also located on the ogee spillway section which divide the overflow crest length into the twenty-three equal 21 feet wide segments on the bays. These piers take the thrust of the radial gates and provide support for the machinery deck. This crest length is considerably longer than the crest of the dam proposed just upstream under Alternate 1. This is primarily due to the submergence effect on the spillway caused by the five downstream bridges, as will be discussed subsequently, and the low head on the spillway.

As in the case of the dam proposed under Alternate 1, the crest of the proposed overflow spillway section was set at Elevation 927.5, 7.5 feet above the probable bottom of the reservoir in this area.

As in the case of the dam proposed under Alternate 1, this selected elevation was a compromise between the desire to provide the maximum gate depth possible so that most of the storage volume in the reservoir would be useable and not wasted as dead storage below the crest of the dam, and the desire to keep the bottom of the gates above any silt that will be deposited behind the dam which could conceivably interfere with the operation of the gates. Setting the crest of the spillway at this elevation will result in a permanent pool (dead storage area) of 740 acre-feet, as previously mentioned.

Both the profile of the upstream and the downstream faces of the concrete overflow spillway section were designed in accordance

with data based on the hydraulic laboratory tests of the U. S. Army Corps of Engineers.[1] The face of the spillway crest downstream from the axis of the dam is defined by the equation:

$$Y = \frac{(X)^{1.776}}{13.298}$$

The face of the spillway crest upstream from the axis of the dam is defined by a single curve with a radius of 5.62 feet with a point of tangency at a distance of 2.49 feet from the axis of the dam. The back face of the overflow spillway section was placed on a slope of 1 : 1 so as to insure stability of the spillway section and to reduce the coefficient of discharge over the spillway as much as possible.

The twenty-three 21 feet x 14 feet radial type gates are positioned on the spillway crest so that the gate sill intersects the face of spillway downstream and approximately one foot below the crest of the spillway, so as to prevent cavitation of the face of the spillway at a partial gate opening. As in the case of the dam just upstream, proposed under Alternate 1, a freeboard of 0.5 feet above the maximum water surface elevation of 940.0 (full pool elevation) has been provided to prevent flow over the top of the gates. The gate trunions have been located just above the water surface of the maximum probable flood nappe, to avoid contact with floating ice and debris, and have been placed below half the depth of the gate above the sill in order to transmit the maximum reaction to the trunion girder as horizontally as possible. Each radial gate is to be operated by a separate hoist and motor located on the machinery deck, so that the gates can be operated singularly or at the same time. The gates will be operated by automatic controls from an operations house, which will be located off the dam in the vicinity of the parking lot and access road, as previously mentioned. Heating equipment will be provided to keep the gates operable during freezing weather.

During the preparation of this report, consideration was given to the installation of vertical lift type gates rather than the

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(1) "Hydraulic Design Charts," Waterways Experiment Station, U.S. Army Corps of Engineers, Vicksburg, Mississippi, 1958-1960.

radial type gates, as shown on the drawings, since the depth of the gates is not very high. However, for purposes of comparison, and because of the many apparent advantages of the radial gates and generally cheaper cost, as previously mentioned, the selection of this type of gate became mandatory for purposes of this report. Further consideration of gate type for this proposed dam on the South Branch of the Thames River will be made during the preparation of the final plans, if this alternate is adopted.

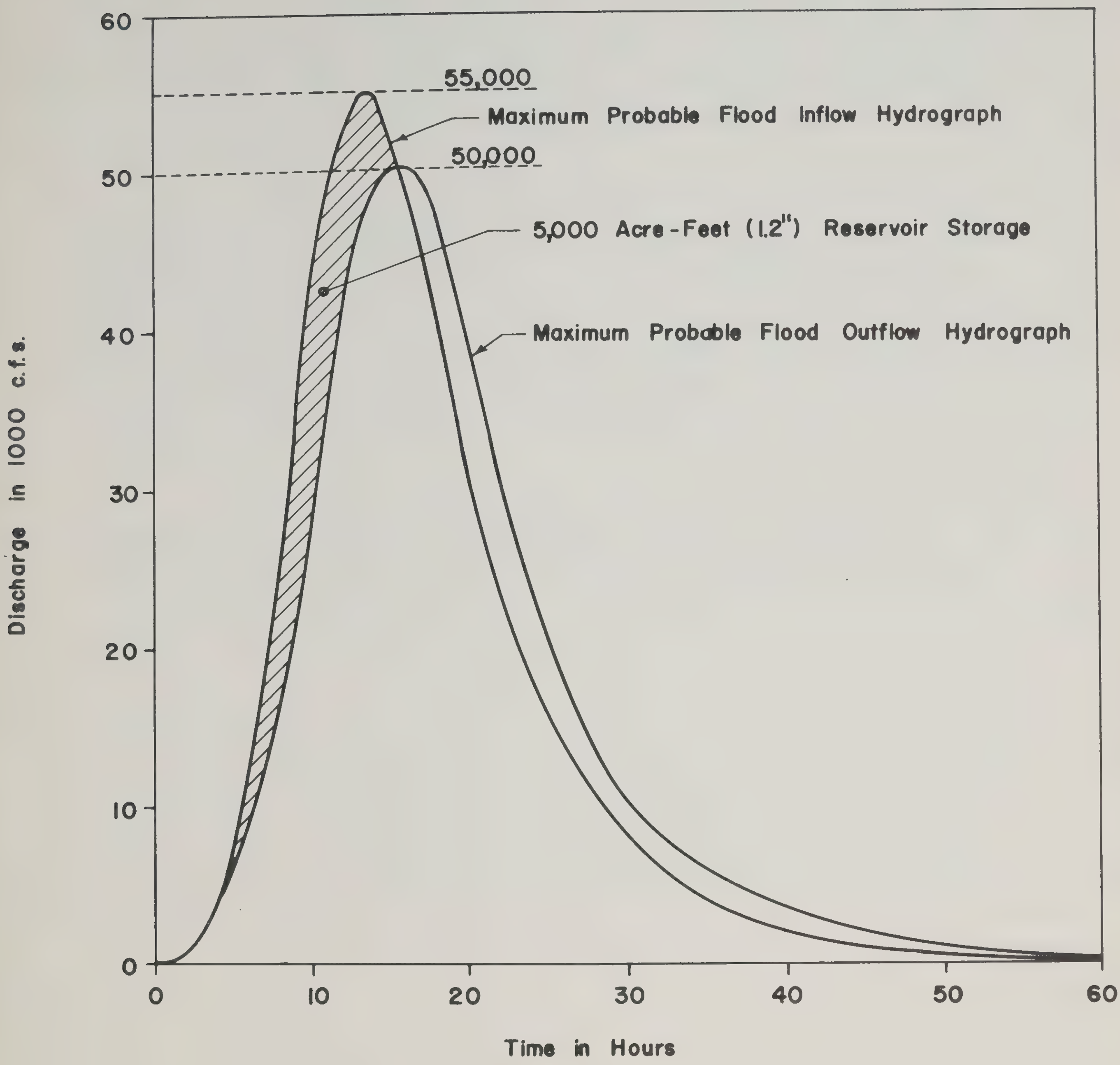
Stop log slots have been provided in the pier and training walls as shown on Drawing No. 25, so that each bay between the piers can be closed in an emergency by the insertion of stop logs. Steel stop logs will be stockpiled off the dam site. No special device will be provided for the placing of the stop logs since it is expected that they will rarely be used, in which case a small truck-mounted crane can be employed.

A 36 inch diameter outlet tube to discharge small rates of flow, such as would be required for discharge from the low flow maintenance pool, will be provided in the south training wall of the concrete spillway section. A 36 inch motor-operated butterfly valve to open and close the tube will be operated from a dry type outlet control chamber adjacent to the south seep wall. The elevation and details of the outlet type will be determined during the preparation of the final plans, if the dams proposed under this Alternate 2 are adopted.

The 483 feet wide overflow spillway crest will be capable of passing 50,000 cfs, the peak discharge of the maximum probable flood (spillway hydrograph) modified by the storage in the reservoir at a maximum reservoir water surface elevation of 940.0, with the gates in the full open position, at a head on the crest of 12.5 feet. The maximum probable flood inflow hydrograph (peak = 55,000 cfs) and the maximum probable flood outflow hydrograph, modified by the storage in the reservoir (peak = 50,000 cfs) as determined by flood routing through the reservoir, is shown on Figure 3.

Tailwater Conditions of Dam Site:

At high rates of flow in the South Branch of the Thames River, ranging from about 35,000 cfs to the peak discharge of the spillway design flood (50,000 cfs), the high head losses through each of the five immediate downstream bridges resulted in a tailwater



MAXIMUM PROBABLE FLOOD
(SPILLWAY INFLOW & OUTFLOW HYDROGRAPHS)
SOUTH BRANCH THAMES RIVER
WOODSTOCK DAM
ALTERNATE 2 — LOW DAMS

FIGURE 3

elevation, at the dam site, higher than the proposed maximum pool elevation of the reservoir. Under conditions existing in the area, the occurrence of a discharge in this range, with its resultant high tailwater elevation, would probably result in the dam structure being overtopped or water sweeping around the south abutment of the dam over the Canadian Pacific Railway Tracks (maximum Elevation 944.0) resulting in probable failure of the dam.

After a thorough study of the situation, the decision was made to place an additional span in the existing Canadian Pacific Railway Bridge at mile 15.3 to reduce the head losses caused by these bridges and thereby make the dam proposed under this alternate feasible. The tailwater rating curve of the dam proposed under this alternate, with an assumed new span in the Canadian Pacific Railway Bridge, is shown on Figure 4. The cost of the placing of this additional new span in the existing bridge has been included in the cost estimate for the dam.

However, even with this additional span in the Canadian Pacific Railway Bridge, the tailwater condition of the dam site is still extremely high. This condition considerably reduces the discharge coefficient of the spillway, which contributes heavily to the extreme length of the spillway crest.

Stilling Basin:

In order to design a stilling basin for the spillway of this proposed dam on the South Branch of the Thames River, the downstream depth of flow at the toe of the spillway (d_1), the velocity of flow at the toe (v_1), the Froude Number (F_1), the conjugate depth (d_2), and the length of the hydraulic jump (L) with no stilling basin in place, were computed at various discharges (Q), and are tabulated in Table 6 below.

TABLE 6

Stilling Basin Characteristics - Dam on South Branch of Thames River

Alternate 2 - Low Dams

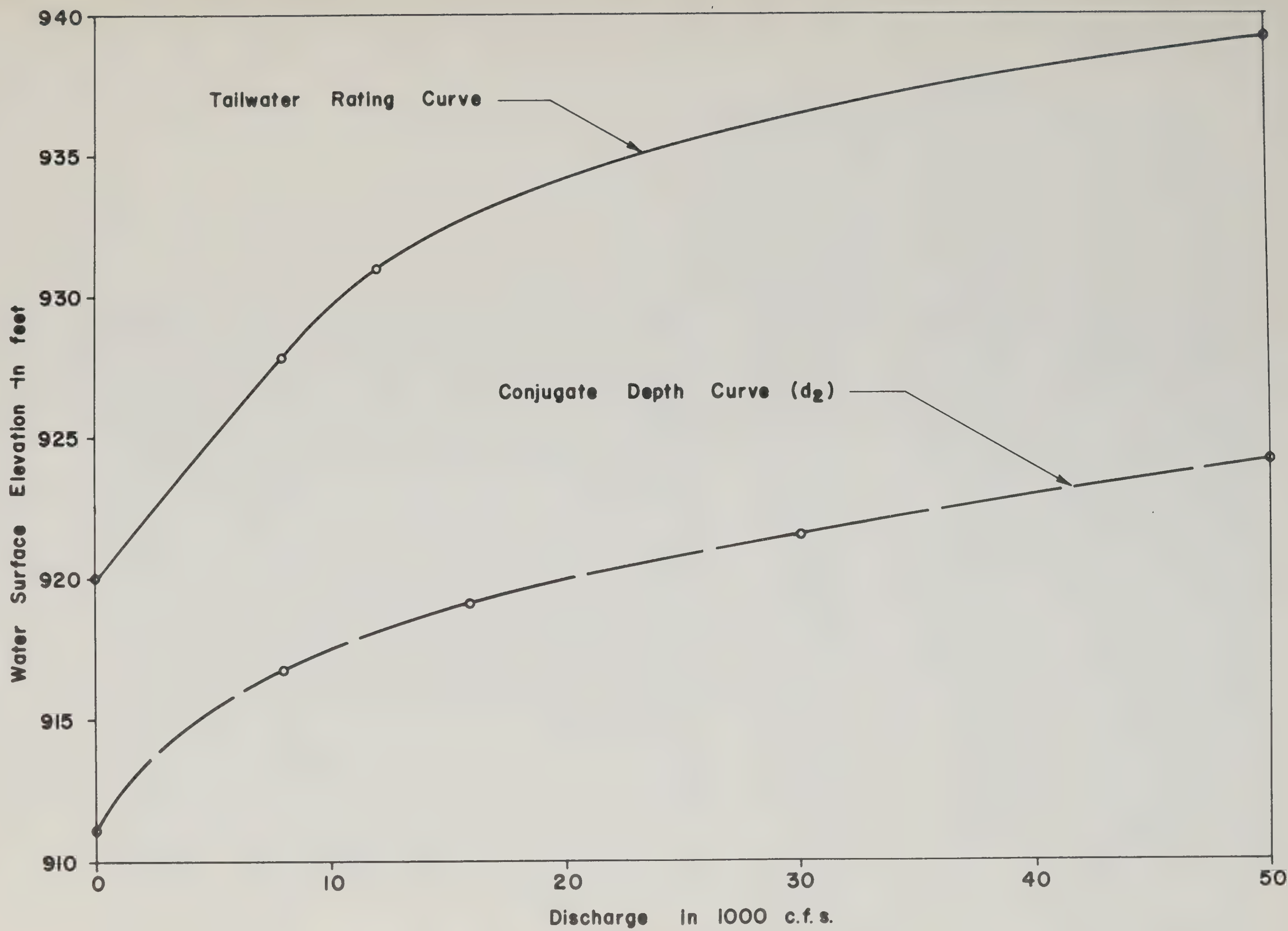
Q (cfs)	d_1 (ft)	v_1 (ft/sec)	F_1	d_2 (ft)	L (ft)
8,000	0.5	35.3	9.1	5.8	35.4
16,000	0.9	35.3	6.4	8.1	49.4
30,000	1.8	34.5	4.5	10.4	61.4
50,000	3.0	33.2	3.3	13.1	72.0

A graph showing the relationship of the conjugate depth curve and the tailwater rating curve for this proposed dam with the top of the stilling basin apron at Elevation 911.0 is shown on Figure 4.

Theoretically, for discharges below 30,000 cfs ($F_1 \geqq 4.5$), a good hydraulic jump should occur and for discharges between 30,000 cfs and 50,000 cfs (F_1 = 4.5 to 3.3) an oscillating type of jump should occur. However, the high tailwater condition shown on the above mentioned figure may completely drown out the jump in both cases, thus, making the action of the jet flowing over the spillway mathematically unpredictable. In such cases, model tests should be made to determine just what happens. However, for purposes of this report, a flat stilling basin apron, with a length equal to the average length of the hydraulic jump throughout the range of discharges and without any chute piers or baffle blocks other than an end sill, was considered a reasonable design for the conditions involved and is shown on the preliminary drawings. Upon selection of the alternate to be adopted, model tests will be made to determine the best type and length of the basin and the desirability of any chute piers or baffles.

Roadway Relocations:

In conjunction with the construction of the proposed low level dam at this site, certain roadways will have to be relocated or raised as described below. Cost estimates for these roadway alterations are included in the items listed in the Cost Estimate section of this report.



TAILWATER RATING & CONJUGATE DEPTH CURVES
WOODSTOCK DAM
ALTERNATE II – LOW DAMS

FIGURE 4

1. The existing Huron Street crossing of the South Branch of the Thames River near the City of Woodstock will be dead ended and the roadway diverted as described for the dam proposed under Alternate 1. The proposed location of the new roadway, on both sides of the river, is the same as that shown on Drawing No. 8 .

2. The grade along a short portion of the existing Innerkip Road (County Road 4) will have to be raised. No new bridge at this location over the Thames River is contemplated.

Utility Relocations:

In conjunction with the construction of a dam on the South Branch of the Thames River, as proposed under this Alternate 2, certain utilities, such as sewers and hydro lines, will have to be relocated as listed and described below:

1. An existing 27 inch combined sewer, owned by the City of Woodstock and located adjacent to and on the south side of the Canadian Pacific Railroad right-of-way, will have to be replaced by a 60 inch pipe and a new overflow chamber, constructed as previously discussed for the dam site proposed under Alternate 1. Under this Alternate 2, the new 60 inch pipe will be placed just south and parallel to the existing 27 inch line (100 feet to the south side of the existing tracks which are to remain in place). A new pipe will also be placed underneath the Canadian Pacific Railway tracks to connect the new overflow chamber with the river. The joints of this new sewer will also be of the rubber gasket type, for the reason as previously given. The proposed 60 inch pipe will be fitted with anti-seep collars and the trench backfilled with impervious material in the vicinity of the dam. The existing 27 inch pipe in the vicinity of the dam will be dug up and the trench filled with impervious material to prevent seepage from the proposed reservoir into the existing pipe, from collecting and possibly saturating and weakening the ground in this area.

The actual amount of new 60 inch pipe under this Alternate 2 is greater than that for the dam proposed under Alternate 1 due to the dam under this Alternate 2 being further downstream.

2. An existing 12 inch O. D. petroleum products pipeline, owned by the Sarnia Products Pipeline Division of the Imperial Oil Company, Ltd. and crossing the South Branch of the Thames River just east (upstream) of Innerkip Road, will have to be replaced by a heavier river-crossing type pipe, at the same location, for a distance of approximately 1,000 feet. Although more desirable, a relocation of this pipe line around the reservoir area was not economically feasible.

3. Five (5) existing telephone poles and 1,800 feet of aerial cable, owned by the Bell Telephone Company of Canada and crossing the South Branch of the Thames River at Huron Street, will be replaced by 1,800 feet of 100-pair cable, since Huron Street will be dead ended and the roadway fill and bridge will be removed.

4. Several telephone poles and lines, owned by the Oxford Telephone Company and crossing the South Branch of the Thames River near Innersip Road (County Road 4), will be reinforced in place or moved.
5. Certain power lines and poles, owned by Ontario Hydro and located at various places throughout the reservoir area, must be relocated.

LOW LEVEL DAM - CEDAR CREEK

General:

Under Alternate 2 this proposed dam structure will be located 1,300 feet upstream (southeast) of Highway 401. This location was selected on the basis that it has the best topographic and physical characteristic of any site in the immediate area.

The maximum water surface elevation of the reservoir (maximum pool elevation) was set at Elevation 950.0, the elevation set in the original Upper Thames Valley Report. This is the highest elevation which would not cause extensive extra flooding of existing farm lands. With the water surface at this elevation, the useable water storage volume created by this dam will be 7,170 acre-feet, which will fully contain the design flood. This is an important factor in the operation of the two dams proposed under this Alternate 2 as described in the section of the report on Reservoir Regulation. An additional 298 acre-feet will exist as permanent dead storage below the crest of the dam. When the water surface is at this maximum elevation of 950.0, a land area of 1,453 acres will be flooded. Storage-elevation and area-elevation curves for this proposed reservoir are shown on Drawing No. 27. A topographic map showing the location of the proposed dam site and reservoir area is shown on Drawing No. 3.

Access to the proposed dam and machinery deck will be by means of a short access road from Highway 59 on the east of the Cedar Creek Valley. A parking lot will be constructed on this side of the valley to provide parking space for employees and visitors. Both the proposed access road and parking lot are shown on Drawing No. 26.

If this dam is constructed, a small operations house and administration building will be constructed on the east side of the valley in the vicinity of the parking lot. The final location of this operations house and administration building will be determined during the preparation of the final plans, if this Alternate 2 is adopted.

Soils and Foundation Conditions:

In order to determine foundation conditions at this dam site, a preliminary soils exploration and rock drilling program along

with hydraulic pressure testing of the underlying rock, was undertaken. The soils exploration and drilling program consisted of placing 6 cased hole borings through the soil overburden to rock and then coring of the rock up to depths of 30 feet. This was followed by hydraulic pressure testing of the rock in certain holes to locate seams which would transmit water under pressure. The soils and rock profile along the centrelines of the proposed dam as determined by the borings and drillings is shown on Drawing No. 4. The soils and foundation condition encountered and the proposed foundation treatment and methods are discussed below.

The soils at the dam site were found to consist of alluvium deposits and of glacial till, overlying a limestone bedrock at a moderate depth. The limestone bedrock was encountered at depths ranging from 50 feet adjacent to the stream to 75 feet at the valley sides.

The glacial till is immediately above the bedrock and varies from 10 feet to 30 feet in thickness. It is a mixture of sand, gravel, clay, (SC, GC and GP) and boulders. Overlying the glacial till is an alluvium deposit composed of layers of clay, and sand (SP, SM, SC, and CL) with a few layers of gravel (GW and GM) and silt (ML). The top 10 feet to 15 feet of the alluvial deposit is predominately granular.

The ground water level was found to be at a depth of zero to 5 feet, except at the east abutment, where it was encountered at a depth of 13 feet. An artesian pressure was found to exist in the glacial till and the bottom layers of the alluvium deposts. The measured head at boring 3 was 9.5 feet above existing ground.

The limestone bedrock is similar in appearance and character to that encountered at the other dam sites.

The borings and undisturbed tests on the clay strata indicate that the foundation conditions are only fair at this site. Based on the three unconfined compression tests, the bearing capacity of the clay was found to be 2.5 to 3.0 tons per square foot. In order to prevent probable excessive settlement of the spillway section in this area the spillway was assumed to be placed on piles. Further investigation may prove this step unnecessary if this dam proposed under Alternate 2 is constructed.

Generally, the alluvial soils will provide an adequate foundation for the earth embankment. Results from one consolidation test on the clay soil indicate that three to six inches of settlement under the embankment can be expected. The embankment should be cambered to provide for this settlement.

It will be necessary to prevent seepage through the upper portion of the alluvium deposit which is quite granular in nature (SP, SW, and SM). It is proposed to use a steel sheet pile cut-off wall extending down into the clay strata to provide a cut-off, and thus prevent seepage through the upper granular portion of the alluvium deposit.

Embankment Section:

The proposed rolled earth portion of the dam is to be of the zoned embankment type with an impervious central core to prevent seepage. A shell of free draining material of high inherent stability will be placed on each side of the core. The embankment sections of this proposed dam are shown on Drawing No. 26. Satisfactory core material in the form of sandy clay (SC) and gravel-sand clay (GC) from glacial till deposits is available throughout the area.

Free draining material in the form of well graded sands, (SW) and sandy gravels (GW) is available for the shell material from pits in the general site of the dam.

Based on the use of the above noted materials for the dam embankment shell and core, a slope of 2-1/4 : 1 was selected for the upstream and downstream faces of the dam for purposes of this report.

The 10 foot width of the top of the impervious core is the minimum dimension which will permit economical placement and compaction of the impervious embankment material. The slopes of the core, 1-1/2 : 1 upstream and 1 : 1 downstream, are to provide an impervious zone with a thickness at the contact of the dam with the foundation of at least 2-1/2 times the height of the dam. This criteria is recommended by the United States Bureau of Reclamation for small dams that do not have a positive cut-off of seepage under the dam.

The proposed embankment will have a crest width of 18 feet and a freeboard of 6 feet (top of the embankment at Elevation 956.0). Rip rap protection will be provided on the upstream slope as protection against wave erosion. Rip rap slope protection will be provided on the downstream slope, up to Elevation 945, to provide slope protection against high tailwater conditions. The remainder of the downstream slope will be sodded. A one-foot thick gravel sand layer (GW) will be provided under the rip rap slope protection to prevent fines from being

washed out through the rip rap by seepage, and maintain the phreatic line within the embankment slope. A toe drain will be installed along the downstream toe of the dam to intercept any seepage.

Spillway Section:

The concrete overflow (ogee) spillway of the proposed dam at this site was placed on the west side of the Cedar Creek Valley so that the mass of flow passing over the spillway would be in line with the existing Highway 401 bridges crossing Cedar Creek, immediately downstream from the dam.

The proposed concrete overflow spillway section will have a total length of 427 feet between the training walls with an overflow crest length of 315 feet on which will be mounted fifteen 21 feet wide by 11.5 feet high radial (Tainter) type gates. Fourteen 8 feet wide piers are also located on the ogee spillway section which divide the overflow crest length into the fifteen equal (21 feet wide) segments or bays. These piers take the thrust of the radial gates and provide support for the machinery deck. This crest length is considerably longer than the crest of the proposed dam on the South Branch of the Thames River under Alternate 1, even though the design discharge is considerably less. This is principally due to the submergence effect of the high tailwater at the front face of the dam, caused by the high head loss through the adjacent downstream Highway 401 bridges which reduces the discharge coefficient, and the low head over the crest of the dam.

The crest of the proposed overflow spillway section was set at Elevation 940.0, 5.0 feet above the probable bottom of the reservoir in this area.

As in the case of the other dams studied, this elevation was a compromise between the desire to provide the maximum gate depth possible so that most of the storage volume in the reservoir would be useable and not wasted as dead storage below the crest of the dam, and the desire to keep the bottom of the gates above any silt that will be deposited behind the dam which could conceivably interfere with the operation of the gates. Setting the crest of the spillway at this elevation will result in permanent pool (dead storage area) of 298 acre-feet as previously mentioned.

The profiles of the upstream and downstream faces of the concrete ogee section were designed in accordance with previously referenced design data which was based on hydraulic laboratory tests of the United States Army Corps of Engineers. The face of the spillway crest downstream from the axis of the dam is defined by the equation:

$$Y = \frac{X^{1.776}}{11.182}$$

The face of the spillway crest, upstream from the axis of the dam, is defined by a single curve with a radius of 4.50 feet with a point of tangency at distance of 1.99 feet from the axis of the dam. The back face of the overflow section was placed on a slope of 1 : 1 so as to provide stability of the spillway section and to reduce the coefficient of the discharge over the spillway.

The fourteen 21 feet x 11.5 feet radial (Tainter) type gates are positioned on the spillway crest so that the gate sill intersects the face of the spillway downstream and approximately 1 foot below the crest of the spillway, so as to prevent cavitation of the spillway face at a partial gate opening. As in the case of the other dams studied, a free-board of 0.5 feet above the maximum water surface elevation of 950.0 (full pool elevation) has been provided to prevent flow over the tops of the gates. The gate trunions have been located just above the water surface of the maximum probable flood nappe to avoid contact with floating ice and debris, and have been placed below half the depth of the gate above the sill, in order to transmit the maximum reaction to the trunion girder as horizontally as possible. Each radial gate is to be operated by a separate hoist and motor located on the machinery deck, so that the gates can be operated singularly or at the same time. The gates will be operated by automatic controls from an operations house, which will be located off the dam in the vicinity of the parking lot and access road as previously mentioned. Heating equipment will be provided to keep the gates operable during freezing weather.

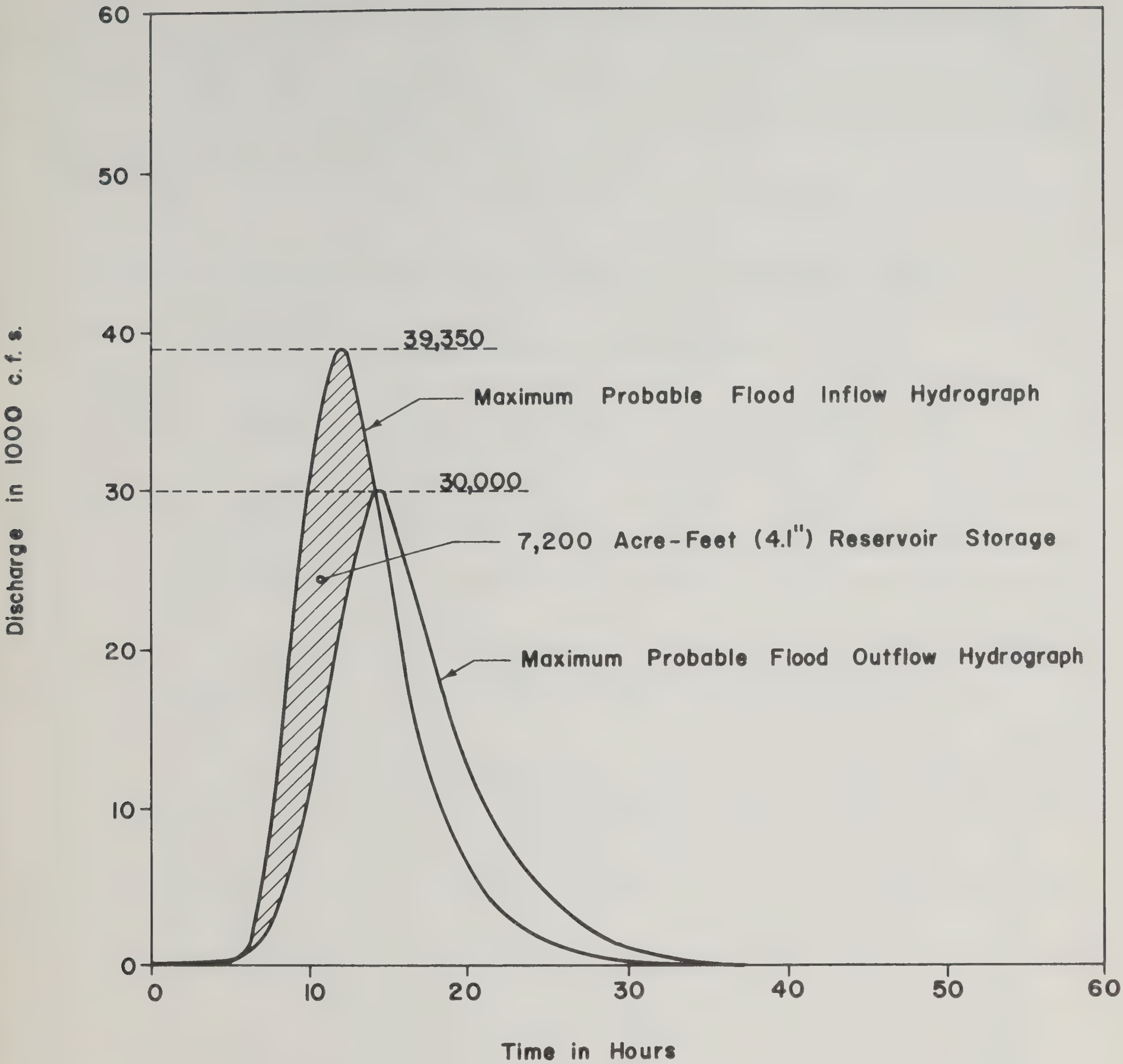
During the preparation of this report, consideration was given to the installation of a vertical lift type gate rather than the radial type gates, as shown on the drawings, since the depth of the gates is not very high. However, for purposes of comparison and

because of the many apparent advantages of the radial gates and generally cheaper cost, as previously mentioned, the selection of the radial type of gate was mandatory for purposes of this report. Further consideration of gate type for the Cedar Creek Dam will be made during the preparation of the final plans if the dams proposed under this Alternate 2 are adopted.

Stop log slots have been provided in the piers and training walls as shown on Drawing No. 27, so that each gate bay between the piers can be closed in an emergency by the insertion of stop logs. Steel stop logs will be stock piled off the dam site. No special device will be provided for the placing of the stop logs because it is expected that they will be rarely used, in which case a small truck-mounted crane can be employed.

A 36 inch diameter outlet tube to discharge small rates of flow, such as would be required for discharge from the low flow maintenance pool, will be provided in the west training wall of the concrete spillway (ogee) section. A 36 inch motor-operated butterfly valve to open and close the tube will be operated from a dry type outlet control chamber adjacent to the west seep wall. The elevation and details of the outlet type will be determined during the preparation of the final plans if the dams proposed under this Alternate 2 are adopted.

The 315 foot wide overflow (ogee) spillway crest will be capable of passing 30,000 cfs, the peak discharge of the maximum probable flood (spillway hydrograph) modified by the storage in the reservoir, at a maximum reservoir water surface elevation of 950.0, with the gates in the full open position and at a head of 10 feet on the crest of the spillway. The maximum probable flood inflow hydrograph (peak = 39,350 cfs) and the maximum probable flood outflow hydrograph, modified by the storage in the reservoir (peak = 30,000 cfs) and as determined by flood routing through the reservoir, is shown on Figure 5. As previously mentioned, the discharge coefficient of the spillway was considerably reduced by submergence, due to a high tailwater condition caused by head losses through the immediate downstream Highway 401 bridges, which resulted in an extremely long spillway length. However, since the proposed dam was not overtopped by this high tailwater condition, no replacement of additional spans in these bridges was considered, for purposes of this report. If this Alternate 2 is adopted, an economic study of the situation should be made before preparation of the final plans.



MAXIMUM PROBABLE FLOOD
(SPILLWAY INFLOW & OUTFLOW HYDROGRAPHS)
CEDAR CREEK DAM
ALTERNATE 2 — LOW DAMS

FIGURE 5

Stilling Basin:

In order to design a stilling basin for the spillway of this proposed dam on Cedar Creek, the downstream depth of flow at the toe of the spillway (d_1), the velocity of flow at the toe (V_1), Froude No. (F_1), the conjugate depth (d_2) and the length of the hydraulic jump (L) with no stilling basin in place, were computed at various discharges (Q) and are tabulated in Table 7 below:

TABLE 7

Stilling Basin Characteristics - Dam on Cedar Creek

Alternate 2 - Low Dams

Q (cfs)	d_1 (ft)	v_1 (ft/sec)	F_1	d_2 (ft)	L (ft)
8,000	0.9	30.6	5.8	6.6	40.0
16,000	1.8	29.6	3.9	8.8	51.0
24,000	2.8	28.4	3.0	10.4	56.0
30,000	3.5	27.5	2.6	11.3	55.0

A graph showing the relationship of the conjugate depth curve and the tailwater rating curve for this proposed dam, with the top of the stilling basin apron at Elevation 928.0, is shown on Figure 6.

Due to the low Froude Number (F_1 = 2.0 to 6.0) at the high discharges, caused principally by the low velocity of flow at the spillway toe which resulted from the low height of the spillway, an oscillating type of hydraulic jump will generally occur. Due to the oscillating position of the flow jet on the apron and the resulting waves and rough surface condition created, the design of an effective stilling basin is difficult and should be determined only by model tests. However, for purposes of this report, a flat stilling basin apron, with a length equal to the average length of the hydraulic jump throughout the range of discharges and without any chute pier of baffle blocks other than an end sill, was considered a reasonable design for the condition involved and is shown on the preliminary drawings. Upon selection of the alternate to be constructed, model tests will be made to determine the best type and length of the basin and the durability of any chute pier and baffles.

Roadway Relocations:

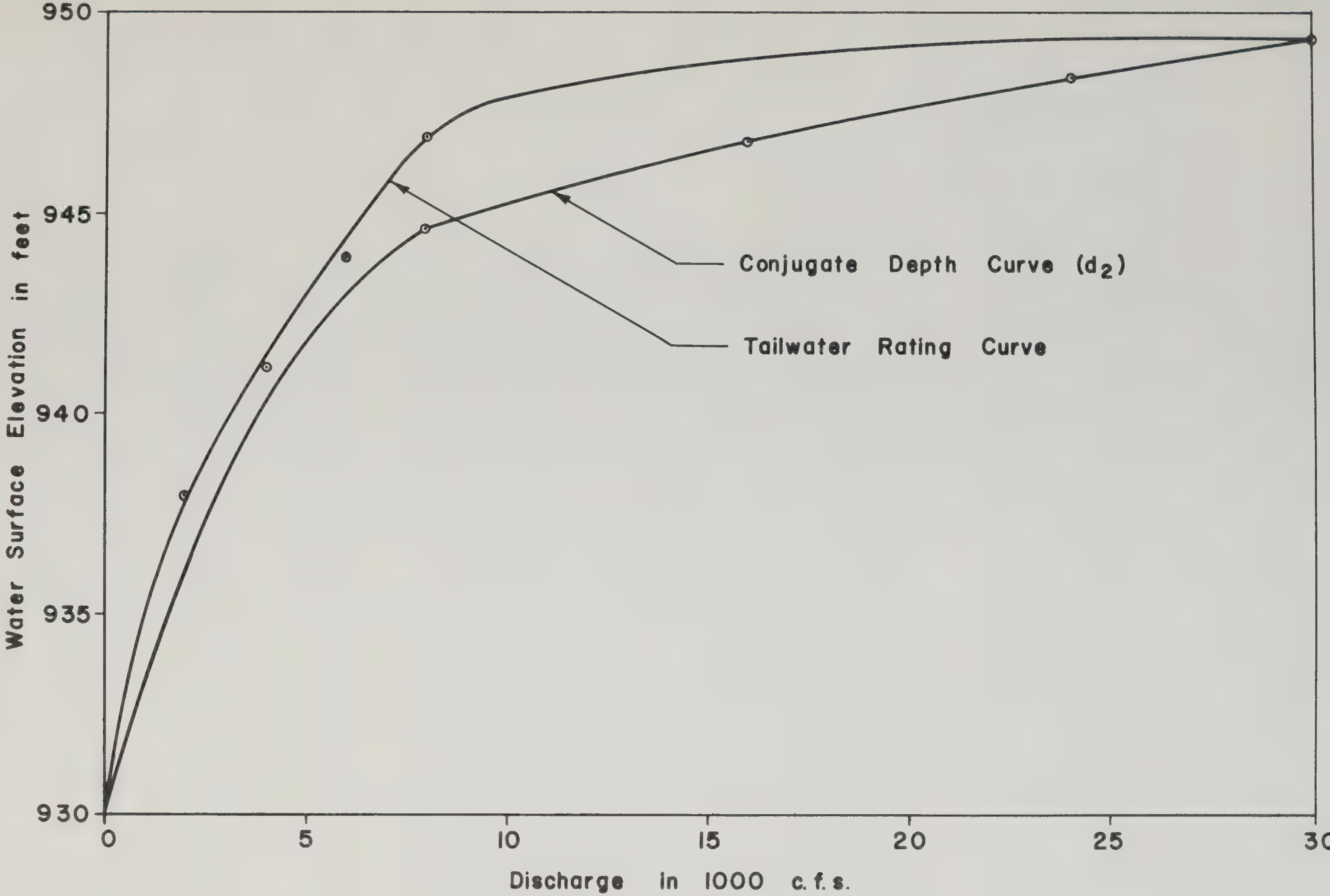
In conjunction with the construction of a dam at this site, various roadways crossing the reservoir area will have to be raised above high water or dead ended. For purposes of this report, the assumption was made that all the roadways would be raised since the quantities involved are small. If this Alternate 2 is adopted, further studies will be made to find out which roadways can be dead ended. Cost estimates for these roadway alterations are included in the Cost Estimates section of this report. The following roadways are required to be relocated:

1. The existing Highway 59 will be flooded out by the proposed maximum pool elevation (Elevation 950.0). Since this road is an important traffic artery, it will be raised above the water surface to a minimum elevation of 955.0. In conjunction with the realignment of this road, a new 120 foot, two span bridge (2 @ 60 feet) will be constructed over Cedar Creek. The details of the bridge are shown on Drawing No. 28. The existing Highway 59 bridge will be demolished or salvaged.

2. The grade along 2,000 feet of the existing Old Stage Road, crossing the proposed reservoir site in Lots 19 and 20 between Concession 3 and 4, in the Township of East Oxford, will have to be raised. A new reinforced concrete bridge will be required to carry this road over Cedar Creek. Because of simplicity and small size, no detailed drawing of this bridge has been included in this report. The existing reinforced concrete bridge at this site will be demolished.

3. The grade along a small portion of the existing Old Stage Road in Lots 16 and 18, Concession 4, in the Township of East Oxford will have to be raised. At both locations, a new bridge or culvert will be required and the existing bridge or culvert removed or demolished.

4. The grade along 3,000 feet of the existing Townline Road between the townships of West Oxford and East Oxford, opposite Lot 21, Concession 5, in the Township of East Oxford, must be raised. A new culvert or bridge will be required and the existing culvert or bridge removed or demolished.



TAILWATER RATING & CONJUGATE DEPTH CURVES
CEDAR CREEK DAM
ALTERNATE 2 - LOW DAMS

FIGURE 6

5. The grade along 2,500 feet of the existing road allowance between Concessions 2 and 3, opposite lots 14 and 15, in the Township of East Oxford, will have to be raised. No new culvert or bridge will be required at this location because the existing culvert is of sufficient size.

Utility Relocations:

In conjunction with the construction of a dam on Cedar Creek, as proposed under this Alternate 2, full pool elevation of 950.0 will require certain utilities such as water mains and hydro lines to be relocated as described below. A cost estimate of these necessary utility relocations is given in the section of this report on Cost Estimates.

1. Certain sections of two existing gravity water mains (one 10 inch concrete and one 12 inch tile), owned by the Woodstock Public Utilities Commission, are within the area to be flooded. Normally, the construction of a dam at this site would require that these water mains be relocated. However, the Woodstock Public Utilities Commission plans to replace these gravity mains in the future with new force mains at a location outside the area to be flooded. Thus, the problem of whether or not to replace these mains becomes a difficult one. The decision was therefore made to assign these existing mains an arbitrary value of $100,000, for the purpose of either replacing the mains or paying the Woodstock Public Utilities Commission an amount equal to the approximate present depreciated value of the mains, which amount would be applied to the cost of the new force main and would be considered as liquidated damages.

2. Certain telephone poles and wires, owned by the Oxford Telephone Company and located in the reservoir area, must be relocated or raised.

3. Certain telephone poles and wires, owned by the Norwich Telephone Company and located in the reservoir area, must be relocated or raised.

4. Certain telephone poles and wires, owned by the Bell Telephone Company of Canada and located in the reservoir area must be relocated or raised.

5. Certain power lines and poles owned by Ontario Hydro and located in the reservoir area must be relocated.

SECTION 7 - CHANNEL IMPROVEMENTS TO THE SOUTH BRANCH OF THE THAMES RIVER AT WOODSTOCK

DESCRIPTION

In order to supplement the flood control effectiveness of the dams proposed under Alternates 1 and 2, channel improvements to the South Branch of the Thames River in the vicinity of Woodstock, Ontario, were studied as part of this report. The locations of these proposed channel improvements are as follows:

1. From a point approximately 6,000 feet downstream from the Governor's Road Bridge (mile 14.3) to the Governor's Road Bridge (mile 15.4). This channel improvement of a portion of the South Branch of the Thames River will be referred to as the Woodstock Channel Improvement for the purposes of this report.

2. From the Governor's Road Bridge (mile 15.4), 4,700 feet upstream to a point opposite the Woodstock Sewage Treatment Plant (mile 16.3) This channel improvement will be referred to as the Woodstock Channel Improvement Extension, for the purposes of this report.

The existing channel in the area of the Woodstock Channel Improvement, although fairly well defined, has extremely limited capacity due to numerous rock shoals which rise up from the bottom of the river, throughout the entire area, and cause excessive backwater even at low flows. The river banks in this area are also heavily overgrown with trees and large roots which project out into the channel and which cause small ice jams during certain spring thaws. Apparently however, very little actual flood damage occurs in this region except for flow backing up into Cedar Creek, which causes flood damage and inconvenience in the City of Woodstock and will be discussed in a subsequent paragraph.

The existing channel in the area of the Woodstock Channel Improvement Extension is poorly defined and meanders throughout the area, due to the extreme flatness of the river in this region. The

numerous oxbows and closed oxbows in this area make the selection of the actual dry weather flow channel difficult to determine. At present, the channel in this area has practically no capacity, so that overbank flow occurs during even the most minor floods or freshets.

The major flood damage in the Woodstock Channel Improvement Region of the Thames River occurs on a low lying section of land on the east side of the Thames River Valley, in the City of Woodstock, along the Cedar Creek tributary channel between the Canadian Pacific Railway Bridge over Cedar Creek and the Mill Street Bridge over Cedar Creek. This low lying area is subject to flooding from two distinct causes as follows:

1. From the backing up of water from the South Branch of the Thames River, when the Thames River is in flood. This low lying land area can also be considered to be in the flood plain of the South Branch of the Thames River.

2. From local floods on the Cedar Creek watershed, which cannot be conducted through this area without overbank flow (flooding), due to the small dry weather flow channel and severe channel constrictions.

This section of the report deals only with the elimination of flooding from the above first cause, the flooding from the second cause being eliminated either by a channel improvement under Alternate 1, or a dam on Cedar Creek to reduce the flow under Alternate 2, both of which are discussed in other sections of this report.

Little or no flood damages occur in the area of the Woodstock Channel Improvement Extension except for some infrequent minor flood damage to Dundas Street west, and one or two adjacent low lying buildings adjacent to Dundas Street west. Almost all of the flood damage in this area is apparently caused by the previously mentioned downstream channel constriction, which will be removed.

In order to ascertain just which areas are subject to flooding at various discharges under conditions as they now exist and the probable causes of such flooding, a series of water surface profiles were computed along the total length of the South Branch of the Thames River from

Beachville to the proposed dam site at Woodstock as is described in detail in Appendix B. These computed water surface profiles for the existing conditions along the South Branch of the Thames River, are shown on Drawings Nos. 32-35. The computation of these water surface profiles, demonstrated that the actual capacity of the existing channel in the region of the proposed Woodstock Channel Improvement without overbank flow occuring, was in the range of only 1,000 to 1,500 cfs. The approximate maximum capacity of the existing Thames River channel and overbank region, with a considerably wide area flooded out and resulting in no damage nor inconvenience to the previously mentioned low lying region in the City of Woodstock, is only about 5,500 cfs.

The maximum elevation of the water surface in the South Branch of the Thames River at the mouth of Cedar Creek that does not result in damage to this low lying area nor inconvenience to the City of Woodstock, is approximately Elevation 920.0, which will be referred to as the damage stage (elevation) for purposes of this report.

PROPORTIONMENT OF THE CHANNEL IMPROVEMENTS

Woodstock Channel Improvement:

The design discharges resulting from the dam(s) proposed under each alternate, as determined by flood routing are listed on line 4 of Table 9 and are shown below.

Alternate 1	-	11,108 cfs
Alternate 2	-	10,152 cfs

These discharges are the maximum peak discharges of the hydrographs marked Alternate 1 - High Dam, and Alternate 2 - Low Dams, as shown on Figure 9. The peak and shape of the hydrographs shown on this figure are greatly influenced by the method or reservoir regulation described in Appendix A.

After thorough study and several trials, the same size trapezoidal channel (100 foot bottom width and 2:1 side slopes) was selected as the size of the proposed channel improvement for both Alternates in this region. This size of channel will keep the water surface elevation below the damage stage elevation of 920.0 at the design discharge. The proposed alignment and cross sections of this proposed channel improvement are shown on Drawing Nos. 29 and 30. At a bottom slope of 0.04%, the channel improvement will carry 6,000 cfs at a depth of 9.8 feet, the approximate bank full stage in this region. This is sufficient capacity to completely contain all minor floods without any overbank flow.

Included in the work proposed with the channel improvement in this area is the removal of all large trees and roots for a distance of 50 feet on both sides of the proposed channel improvement, to prevent the possibility of ice and debris jamming and causing backwater during times of overbank flow at high discharges. It is also proposed that the earth portions of both sides of the channel be rip rapped to prevent erosion and silting of the channel.

The most difficult feature of the work in this area will be the removal of the rock in the channel, in the vicinity of the abutments and pier of the existing Canadian National Railway Bridge. A typical section of the proposed channel in the vicinity of the bridge is shown on Drawing No. 29.

Woodstock Channel Improvement Extension:

The design discharges resulting from the dam(s) proposed under each alternate, as determined by flood routing, are listed on line 5 of Table 9 and are shown below.

Alternate 1 - 4,000 cfs.
Alternate 2 - 10,150 cfs.

These discharges are the maximum peak discharges of the hydrographs designated Alternate 1 - High Dam and Alternate 2 - Low Dams, as shown on Figure 10. The peak and slope of these hydrographs are also greatly influenced by the method of reservoir regulation described in Appendix A.

After thorough study of the area involved and the principal causes of major flooding in the area at high flows (the three immediate downstream bridges), a trapezoidal channel with a bottom width of 75 feet and 2:1 side slopes, at a slope of 0.875% was selected as the maximum possible size of the channel improvement in this region. This 75 foot bottom width trapezoidal channel will just about carry 6,000 cfs at the approximate bank full depth of about 5 feet in this area. Any larger channel would not carry more flow since this area would be drowned out by the head losses through the downstream bridges at the higher discharge rates.

This channel size will generally carry all the minor floods, but some overbank flow will occur at the Alternate 2 design discharge indicated above. For Alternate 1, a trapezoidal channel with a 50 foot bottom width and 2:1 side slopes at the same channel slope of 0.875% was selected as the size of the Woodstock Channel Improvement Extension. A channel of this size will carry the project design discharge of 4,000 cfs at a depth of 5 feet without overbank flow.

Under both Alternates, in order to make the flood plain land in this region useable for recreational purposes, the new channel will be placed on the western side of the valley. Under both Alternates, both sides of the channel will be rip rapped to prevent erosion of the sides and to hold the channel in position. Also all trees and roots will be removed for a distance of 50 feet on both sides of the proposed channel improvement.

The earth and rock excavated for the proposed channels of the Woodstock Channel Improvement and Woodstock Channel Improvement Extension will be completely moved off the flood plain, except for filling in the old channel in the Woodstock Channel Improvement Extension area, in order to preserve the original flood plain storage. In addition, no dykes or levees will be placed in these areas, since they would eliminate natural storage.

SECTION 8 - WATER CONSERVATION IN THE DRAINAGE BASIN

DESCRIPTION

The reservoir(s) proposed under the two Alternates are intended for multi-purpose use and will normally be kept full or as full as possible to provide water for each of the various conservation purposes. The tops of the gates of the dam(s) proposed under each Alternate have been placed as high as possible to secure the maximum amount of conservation storage possible. The maximum full pool elevation when passing the maximum probable flood is coincident with the static full pool elevation. The resultant maximum total amounts of water available for conservation purposes, as proposed under each Alternate, are listed in Table 8 as follows.

Table 8

Water Available for Conservation Above Dam Crests

Alternate 1	
Dam on South Branch of Thames River	13,400 acre-feet
Alternate 2	
Dam on South Branch of Thames River	5,000 acre-feet
Dam on Cedar Creek	7,200 acre-feet
Total	12,200 acre-feet

LOW FLOW MAINTENANCE AND WATER POLUTION

At the present time, only small amounts of raw sewage are dumped into the South Branch of the Thames River. The main sources of this raw sewage are the various small creameries and cheese factories and light industries between Ingersoll and Woodstock, which discharge their waste materials directly into the river. Another minor source of pollution is from the quarries in the Beachville area which pump various types and degrees of waste from the quarry floors directly into the channel improvement area between Beachville and Ingersoll. However, the pollution resulting from these sources is not overly serious and during the next several years will probably be completely eliminated.

Generally, the sewage treatment plants at Woodstock and Ingersoll effectively treat the sewage so that the B. O. D. and Coliform Counts of the sewage plant effluents are quite low. However, no sewage treatment system is 100% effective in treating sewage water so that a certain minimum rate of flow of the receiving waters is required, to effectively dilute the sewage plant effluent. Generally, the Ontario Water Resources Commission requires a dilution ratio of 6 to 1 between the flow in the channel and the effluent from the sewage treatment plant.

As can be seen from this ratio, the necessary quantity of flow in the river is dependant upon the amount of sewage being discharged into the river from the sewage treatment plant. This is an important factor at the City of Woodstock, where the average rate of sewage flow (effluent from the sewage treatment plant) at the present time averages approximately a steady 9. 1 cfs. Using the dilution ratio of 6, the average rate of flow in the South Branch of the Thames River should be 54 cfs.

The average stream flow in the South Branch of the Thames River at Woodstock is approximately 100 cfs (5% probability) which apparently should be sufficient to maintain the required dilution ratio. However, much of this average flow occurs during the spring thaw, resulting in extended periods of time during most years when there is not sufficient flow to maintain the dilution ratio. During these periods of low flow, which generally occur during the summer months, the flow in the South Branch of the Thames River can drop to an average rate of about 19 cfs (5% probability) resulting in a difficiency

of flow of 35 cfs in the river. It is during these periods of time when the river in the vicinity of the City of Woodstock becomes quite polluted and odouriferous.

In order to provide an additional flow of 35 cfs for the minimum recommended period of 240 days, a total storage volume in the reservoir (low flow maintenance pool) of 17,000 acre-feet would be required. For a 200 day low flow period, a total volume of 14,000 acre-feet would be required. A total storage volume of only 13,400 acre-feet is available under Alternate 1, and 12,200 acre-feet under Alternate 2, as previously mentioned.

However, at the present time, the City of Woodstock is energentically engaged in a campaign to cut down infiltration into the city sanitary sewer system, the principal cause of the high rate being discharged from the sewage treatment plant. If fully pursued and carried through, the Ontario Water Resources Commission is of the opinion that the effluent from the treatment plant can be reduced to a rate of 4.5 cfs. This would reduce the required stream flow during the low flow periods to 27 cfs using the dilution ratio of 6, thereby reducing the stream flow deficiency during these periods to 8 cfs. In order to provide this additional flow of 8 cfs for the minimum recommended 240 day minimum period, a total storage volume in the low maintenance pool of 3,900 acre-feet (say 4,000) would be required.

The discussion and computation made above were based on present day population figures which may increase over a period of years, thereby increasing the sewage plant effulent and the resultant necessary size of the low flow maintenance pool.

Based on the above and on the assumed success of the program to reduce infiltration into the sanitary sewer system, a total of 4,000 acre-feet of the total reservoir storage volume available under either Alternate was conditionally assigned as a low flow maintenance pool. The assignment of this amount of storage volume for low flow maintenance has received the approval of the Ontario Water Resources Commission.

If, during an extended dry period, this entire low flow maintenance pool is used to augment downstream flow, the water surface elevation of the high dam proposed under Alternate 1 would

drop down 3.8 feet to Elevation 946. 2 if the reservoir was full (Elevation 950. 0) at the start of the dry period. If 2, 000 acre-feet of each of the low dams proposed under Alternate 2 were designated a low flow maintenance pool, the water surface of the dam on the South Branch of the Thames River would drop 3. 3 feet to Elevation 936. 7 and the water surface of the dam on Cedar Creek would drop 1. 4 feet to Elevation 948. 6 if the resevoir in each case was full at the start of the dry period.

OTHER CONSERVATION USES

After deducting the necessary 4,000 acre-feet for a low flow maintenance pool, and ignoring any losses due to evaporation, substantial quantities of storage (9,400 acre-feet under Alternate 1, and 8,200 acre-feet under Alternate 2) would still be available for other conservation uses, such as recreation and irrigation. As an alternate, a large portion of this remaining storage could be permanently assigned as flood control storage or could be assigned to flood control storage for a part of the year (e. g. during February and March to impound the snowmelt runoff). The usage assigned to this remaining storage under either Alternate should be made only after a careful study by the Upper Thames River Conservation Authority.

SECTION 9 - FLOOD CONTROL EFFECTIVENESS

If there is sufficient storage volume available to reasonably control a drainage basin, the actual effectiveness of a dam or group of dams in reducing downstream flood peaks, depends almost entirely on how the gates are operated. For purposes of this report, the reservoir(s) proposed under each Alternate were assumed to be regulated in accordance with a certain set criteria of operation, which are fully derived and explained in the section on Reservoir Regulation in Appendix A.

The effect of each of the proposed Alternates in controlling and reducing the peak of the project design flood at various locations along the South Branch of the Thames River and Cedar Creek, as determined by flood routing, is shown on Figures 7 through 11. The peak flows of the hydrographs under existing conditions and as would be proposed under each Alternate are listed in Table 9. The double peaks of the hydrographs resulting from both alternates, as shown on these figures is a direct result of the reservoir regulation and operation procedure described in Appendix A.

No design flood hydrograph is shown for the existing condition at Ingersoll (Mile 6.4) on Figure 7 since the peak of this design flood hydrograph is sufficiently high causing certain sections of the existing channel improvement to be overtopped resulting in much of the flow spilling into the quarries of that region, thus making the computations of the design hydrograph for the existing condition impossible to compute. The peak flow of the design flood hydrograph at Beachville (existing conditions) at the start of this channel improvement is 19,500 cfs compared with a channel capacity of 14,000 cfs at this location.

TABLE 9

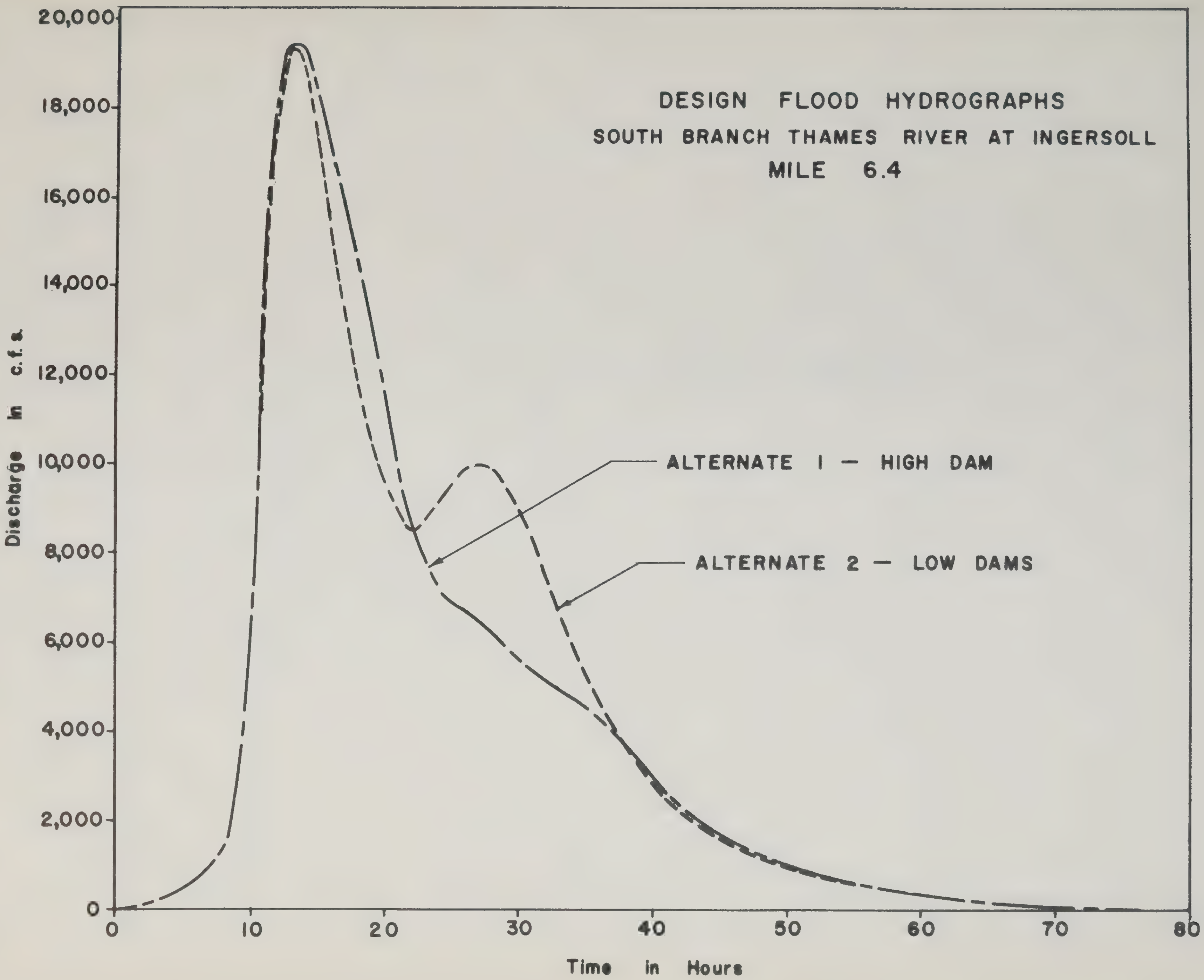
Peak Flow Of Project Design Floods - Existing And Proposed Conditions - South Branch Of The Thames River

Location	Alternate I (c. f. s.)	Alternate II (c. f. s.)	Existing (c. f. s.)
Ingersoll, Mile 6.4	19,412	19,250	--
Beachville, Mile 10.8	11,631	10,047	19,529
Beachville N., Mile 13.0	11,920	10,030	19,500
Governor's Rd. Br., 15.4	11,108	10,152	19,315
Woodstock Dam Site, 17.3	4,000	10,500	14,950

Generally, the existing channel improvement in the vicinity of the quarries and the Town of Ingersoll will fully contain the peak of the routed Design Flood Hydrograph if the dams proposed under either alternate are constructed. However, there is a short section between the start of the channel improvement at Mile 5.5 and the tributary joining the river just upstream from the Thames Street Bridge at Mile 6.5 where the capacity of the channel is only 14,000 cfs (10,000 cfs at the existing bridge) whereas the peak of the Design Flood is approximately 19,000 cfs for both alternates as shown on Figure 7 and indicated on Table 9.

This high peak is principally caused by the runoff from the fairly large steep sub-drainage basin contributary to the main stem in this area. The local inflow hydrograph for this area is shown on Figure 25, titled "Design Flood Hydrograph, South Branch Thames River, Local Inflow, Mile 6.4 to Mile 10.8 (Ingersoll to Beachville).

In order to correct this situation, consideration should be given to construction of a small flood water retarding dam or dams on the above mentioned main tributary of this sub-drainage basin in order to control the contributary area, or by the replacement of the existing inadequate bridges in the area. A good retarding dam site is on the main tributary on the South side of the Thames River Valley near the Sweaburg Road.



DESIGN FLOOD HYDROGRAPHS
SOUTH BRANCH THAMES RIVER AT INGERSOLL
MILE 6.4
ALTERNATE I — HIGH DAM
ALTERNATE 2 — LOW DAMS
Discharge in c.f.s.
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0
10
20
30
40
50
60
70
80
Time in Hours

FIGURE 7

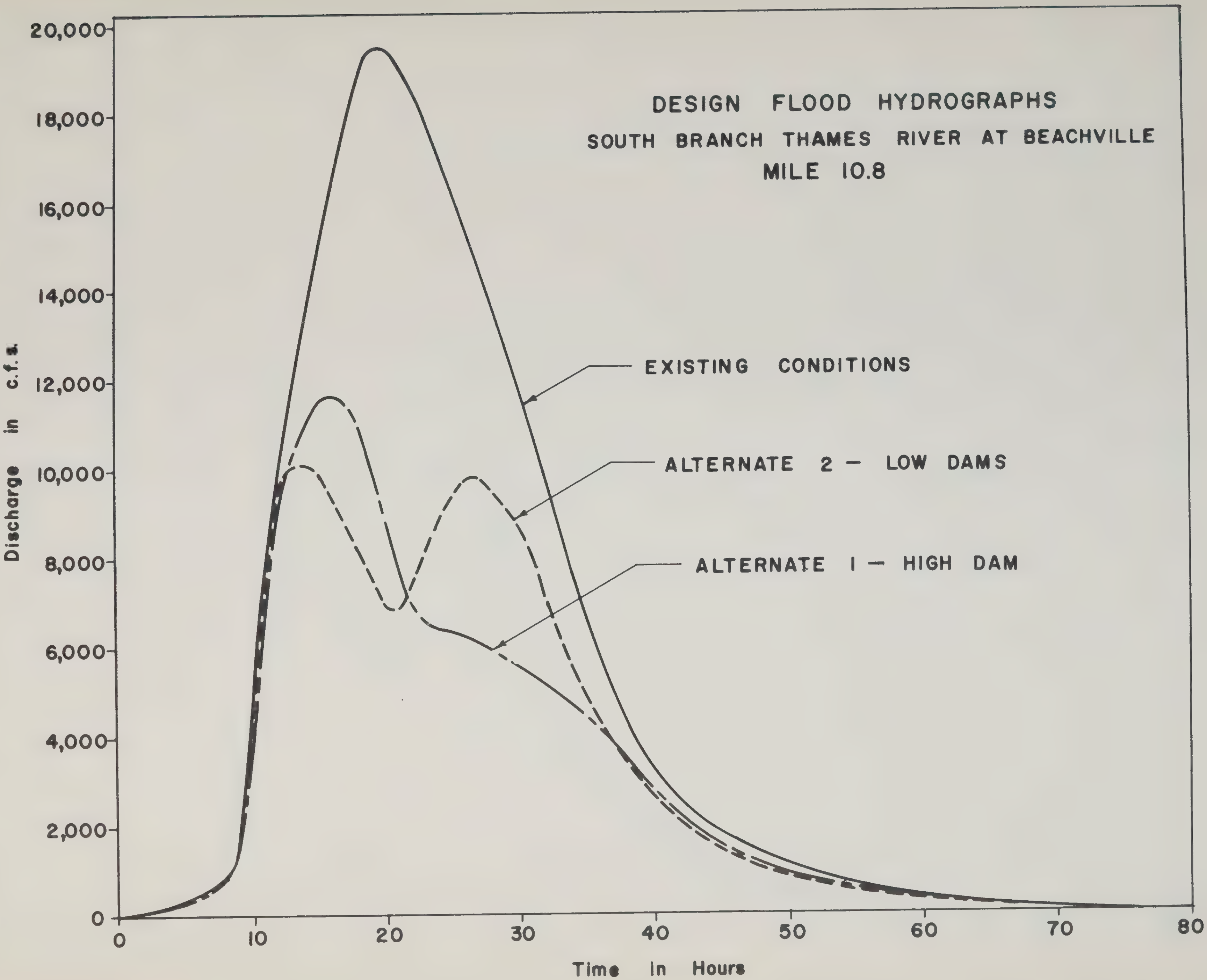

DESIGN FLOOD HYDROGRAPHS
SOUTH BRANCH THAMES RIVER AT BEACHVILLE
MILE 10.8
EXISTING CONDITIONS
ALTERNATE 2 — LOW DAMS
ALTERNATE 1 — HIGH DAM
Discharge in c.f.s.
Time in Hours
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0
10
20
30
40
50
60
70
80

FIGURE 8

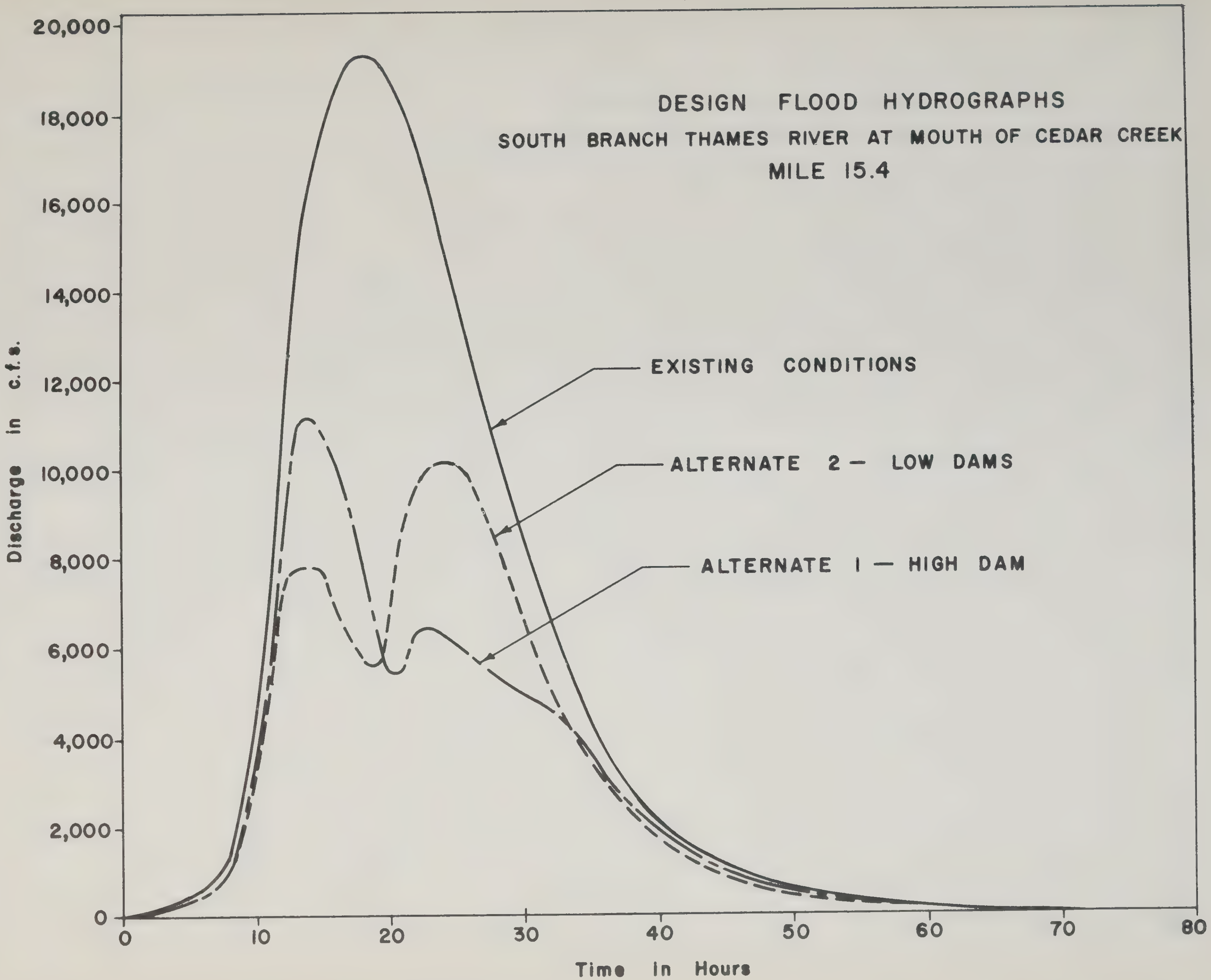

DESIGN FLOOD HYDROGRAPHS
SOUTH BRANCH THAMES RIVER AT MOUTH OF CEDAR CREEK
MILE 15.4
EXISTING CONDITIONS
ALTERNATE 2 — LOW DAMS
ALTERNATE 1 — HIGH DAM
Discharge in c.f.s.
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0
10
20
30
40
50
60
70
80
Time in Hours

FIGURE 9

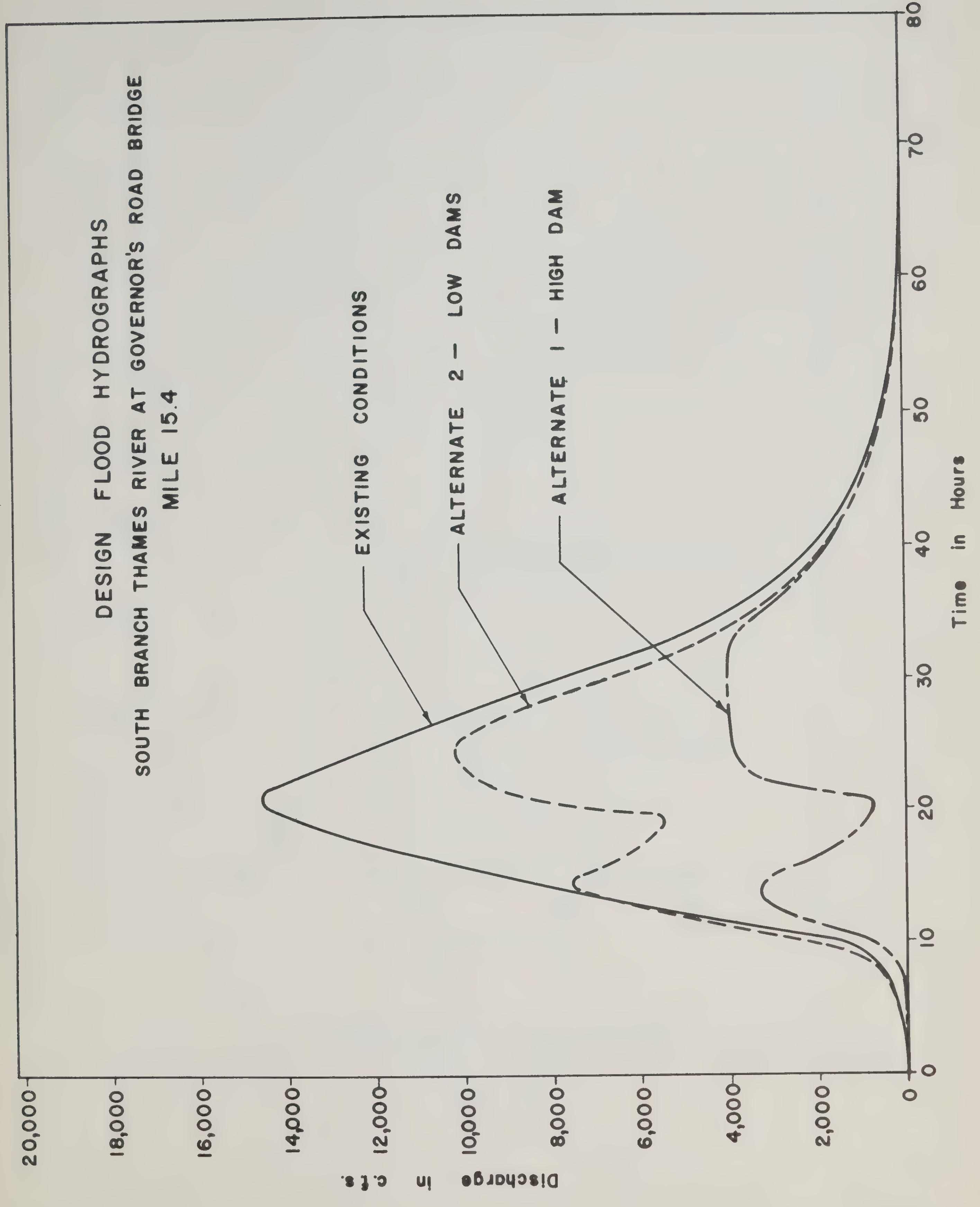
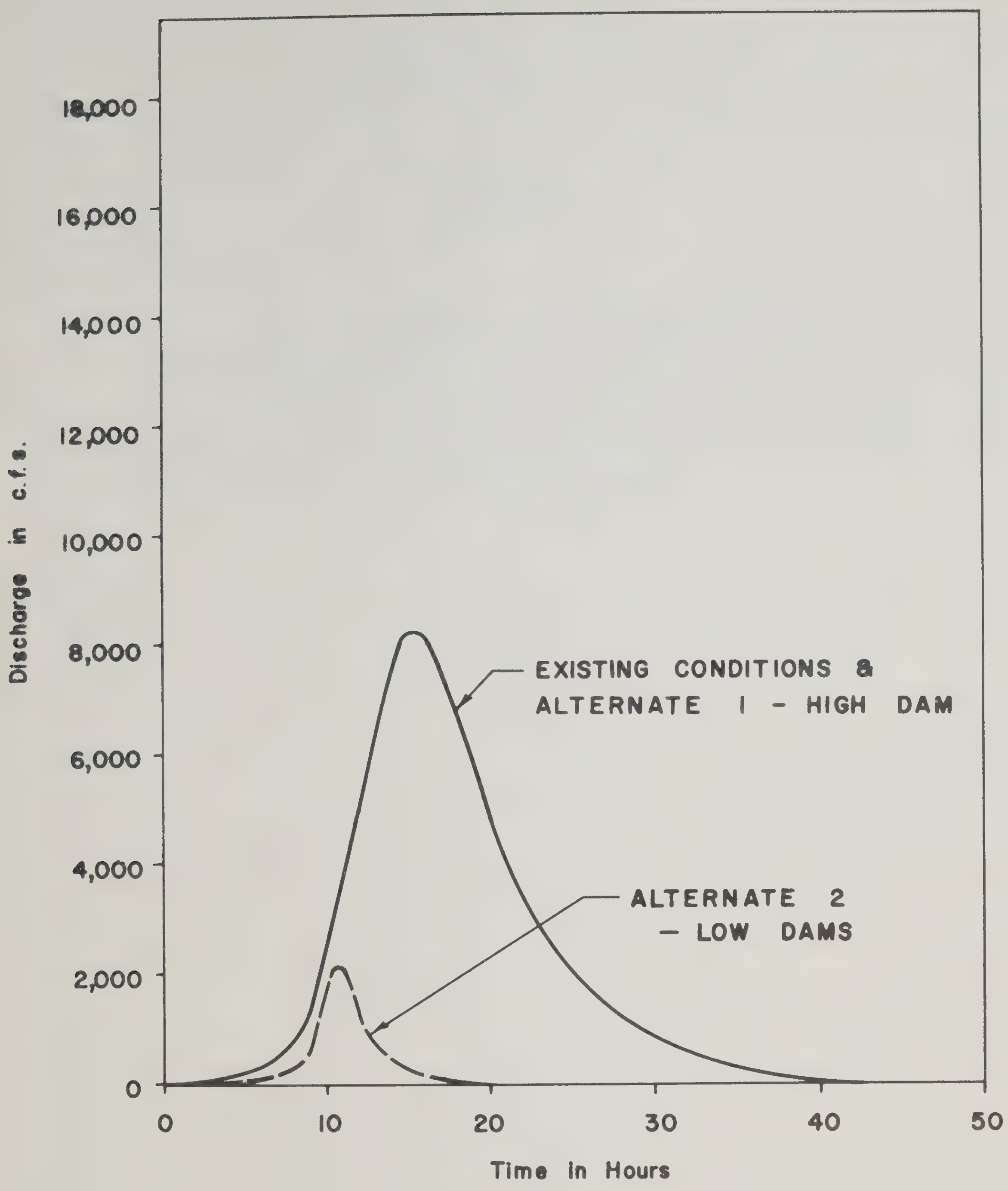

DESIGN FLOOD HYDROGRAPHS
SOUTH BRANCH THAMES RIVER AT GOVERNOR'S ROAD BRIDGE
MILE 15.4
EXISTING CONDITIONS
ALTERNATE 2 — LOW DAMS
ALTERNATE 1 — HIGH DAM
Discharge in c.f.s.
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
Time in Hours
0
10
20
30
40
50
60
70
80

FIGURE 10



DESIGN FLOOD HYDROGRAPHS
CEDAR CREEK AT ENTRANCE TO SOUTH BRANCH THAMES RIVER
MILE 0.0

FIGURE 11

SECTION 10 - COST ESTIMATES

GENERAL

The cost estimates for the dam(s) proposed under both alternates, and the channel improvements were computed in the standard manner by multiplying the various quantities of the materials by reasonable unit prices for each quantity. The computed costs for the proposed dam(s) and channel improvements are given on the following pages.

ALTERNATE I - HIGH DAM

High Level Dam on S. B. Thames River (Including Relocation of the Canadian Pacific Railway):

Item	Description	Cost
1.	Clearing Reservoir Site & Demolition	$ 104,500
2.	Earth work & Rip Rap	474,100
3.	Preparation of Foundation & Grouting	134,400
4.	Concrete Overflow Spillway Section	648,700
5.	Gates, Hoisting Equipment & Appurtances	171,600
6.	Operations & Administration Building	8,000
7.	Highway Bridges & Culverts	131,300
8.	Highway Paving & Drainage	91,000
9.	Railway Bridges & Culverts	247,800
10.	Railway Tracks, Ballast & Row Fence	288,500
11.	Railway Signal & Communications	70,000
12.	Utility Relocations & Alterations	285,600
13.	Electrical Work & Emergency Power	12,000
	TOTAL CONSTRUCTION COSTS	$2,667,500
	Engineering, Testing & Legal	266,700
	Land Costs & Contingencies	650,000
	TOTAL COST	$3,584,200

Cedar Creek Channel Improvement:

Item	Description	Cost
1.	Clearing Channel & Demolition	$ 9,000
2.	Earthwork & Rip Rap	24,000
3.	Bridges & Culverts	172,200
4.	Highway Paving & Drainage	17,200
5.	Utility Relocations & Alterations	27,100
	TOTAL CONSTRUCTION COST	$ 249,500
	Engineering, Testing & Legal	25,000
	Land Cost & Contingencies	25,000
	TOTAL COST	$ 299,500

ALTERNATE II - LOWDAMS

Low Level Dam on S.B. Thames River:

Item	Description	Cost
1.	Clearing Reservoir Site & Demolition	$ 57,500
2.	Earthwork & Rip Rap	217,400
3.	Preparation of Foundation & Grouting	101,900
4.	Concrete Overflow Spillway Section	1,410,800
5.	Gates, Hoisting Equipment & Appurtances	426,100
6.	Operations & Administration Building	8,000
7.	Highway Bridges & Culverts	102,000
8.	Highway Paving & Drainage	39,200
9.	Utility Relocations & Alterations	160,500
10.	Electrical Work & Emergency Power	10,000
	TOTAL CONSTRUCTION COSTS	$2,533,400
	Engineering, Testing & Legal	253,300
	Land Cost & Contingencies	400,000
	TOTAL COST	$3,186,700

Low Level Dam on Cedar Creek:

Item	Description	Cost
1.	Clearing Reservoir Site & Demolition	$ 114,800
2.	Earthwork & Rip Rap	100,500
3.	Preparation of Foundation & Sheet Piling	257,000
4.	Concrete Overflow Spillway Section	708,400
5.	Gates, Hoisting Equipment & Appurtances	232,700
6.	Operations & Administration Building	8,000
7.	Highway Bridges & Culverts	95,200
8.	Highway Paving & Drainage	68,700
9.	Utility Relocations & Alterations	121,300
10.	Electrical Work & Emergency Power	10,000
	TOTAL CONSTRUCTION COSTS	$1,716,600
	Engineering, Testing & Legal	171,700
	Land Costs & Contingencies	400,000
	TOTAL COST	$2,288,300

CHANNEL IMPROVEMENTS TO THE SOUTH BRANCH OF THE THAMES RIVER

Woodstock Channel Improvement:

Item	Description		Cost
1.	Clearing & Grubbing	$	2,400
2.	Earthwork		98,500
3.	Slope Protection (Rip Rap)		20,700
	TOTAL CONSTRUCTION COST	$	121,600
	Engineering, Testing & Legal		12,200
	Land Costs & Contingencies		2,000
	TOTAL COST	$	135,800

Woodstock Channel Improvement Extension (Alternate I):

Item	Description		Cost
1.	Clearing & Grubbing	$	800
2.	Earthwork		38,100
3.	Slope Protection (Rip Rap)		27,700
	TOTAL CONSTRUCTION COST	$	66,600
	Engineering, Testing & Legal		6,700
	Land Costs & Contingencies		1,500
	TOTAL COST	$	74,800

Woodstock Channel Improvement Extension (Alternate II):

Item	Description		Cost
1.	Clearing & Grubbing	$	800
2.	Earthwork		50,800
3.	Slope Protection (Rip Rap)		27,700
	TOTAL CONSTRUCTION COST	$	79,300
	Engineering, Testing & Legal		7,900
	Land Costs & Contingencies		1,500
	TOTAL COST	$	88,700

COST ESTIMATES SUMMARY

ALTERNATE I -

1.	High Level Dam on South Branch of Thames River (Including Relocation of the Canadian Pacific Railway)	$3,584,200
2.	Cedar Creek Channel Improvement	299,500
	TOTAL COST	$3,883,700

ALTERNATE II -

1.	Low Level Dam on South Branch of the Thames River	$3,186,700
2.	Low Level Dam on Cedar Creek	2,288,300
	TOTAL COST	$5,475,000

Channel Improvements to the South Branch of the Thames River

1.	Woodstock Channel Improvement:	$ 135,800
2a.	Woodstock Channel Improvement Extension (Alternate I):	$ 74,800
2b.	Woodstock Channel Improvement Extension (Alternate II):	$ 88,700

SECTION 11 - ADVANTAGES OF ALTERNATE 1

1. The ultimate total cost of the work proposed under Alternate 1 is considerably less than the ultimate total cost of the work proposed under Alternate 2.

   Alternate 1 Costs

1.	Dam on South Branch of Thames River, including Railroad relocation	$3,584,200
2.	Cedar Creek Channel Improvement	299,500
	Total Cost -	$3,883,700

   Alternate 2 Costs

1.	Dam on South Branch of Thames River	$3,186,700
2.	Dam on Cedar Creek	2,288,300
	Total Cost -	$5,475,000

   Each of the above alternates would be supplemented by the construction of the Woodstock Channel Improvement and the Woodstock Channel Improvement Extension.

2. The cost of operating and maintaining the single high level dam proposed under Alternate 1, will be considerably less than the cost of operating and maintaining the two dams proposed under Alternate 2.

3. Under Alternate 1, the full low flow maintenance pool will be located upstream from the Woodstock sewage treatment plant, thus insuring the region between the sewage treatment plant and Cedar Creek of a longer period of diluted flow during dry periods. The proximity of this region to the City of Woodstock makes this particularly important.

4. More water for conservation purposes is available in the reservoir proposed under Alternate 1 (13,400 acre-feet) than would be available in the two reservoirs proposed under Alternate 2 (12,200 acre-feet).

5. Under Alternate 1, considerably less land will be taken out of Agricultural production (1,000 ± acres) than will ultimately be taken out of if the two dams proposed under Alternate 2 are constructed (1,400 ± acres). Therefore, the economic impact (loss of income) on Oxford County will be less.

6. If the single dam proposed under Alternate 1, is constructed, less land will be removed from the tax assessment rolls (2,375 acres) than for the two dams proposed under Alternate 2 (3,546 acres).

7. The life of the single high dam proposed under Alternate 1, under the impact of sedimentation, will be longer than the life of the two lower dams proposed under Alternate 2.

8. Less evaporation will occur off the water surface (1,131 acres, at maximum full pool) of the single high dam proposed under Alternate 1 than will occur off the combined water surfaces (2,093 acres at maximum full pool) of the two dams proposed under Alternate 2, thereby insuring more water for various conservation purposes.

9. The foundation conditions at the site of the Cedar Creek dam proposed under Alternate 2 are quite poor due to rock being at a greater depth than in Alternate 1 which could result in considerable seepage through the earth and sheet pile retaining wall underneath the dam.

10. The single high dam on the South Branch of the Thames River as proposed under Alternate 1, with it's greater storage concentrated on the main stem, will be easier to operate and regulate.

11. Under Alternate 1, the City of Woodstock will benefit from the proposed new Main Street Bridge over the Cedar Creek channel improvement which will replace an existing narrow inadequate bridge.

12. Under Alternate 1, less money will have to be spent to improve the channel of the South Branch of the Thames River between Governor's Road Bridge and the Woodstock Sewage Treatment (Woodstock Channel Improvement Extension).

13. Under Alternate 1, certain benefits such as land enhancement will accrue to the City of Woodstock as a result of the relocation of the Canadian Pacific Railway to the north side of the Thames River Valley.

14. Since there are only a few good dam sites in the Thames River Valley, it is always preferrable to fully utilize a dam site to its optimum. In other words, the higher the dam, as under Alternate 1, the better utilization of the dam site will occur. Also, in this particular instance, if Alternate 1, were adopted, the Cedar Creek Dam site could be saved for some future use or purpose.

15. Under Alternate 1, no new span would be required in the existing Canadian Pacific Railway Bridge over the South Branch of the Thames River at Woodstock. Since this is a fairly heavily travelled main line, the addition of a new span would be difficult.

16. Under Alternate 2, there will be a large surplus of excavated material at both dam sites which will be difficult to dispose of.

SECTION 12 - ADVANTAGES OF ALTERNATE 2

1. Under Alternate 2, water for conservation purposes would be stored and available at two different places. If stored water were used for irrigation, a much larger area could receive benefits from such a use. Also, there would be recreational benefits in two separate areas.

2. Under Alternate 2, a mechanical breakdown of one gate, during a flood, would not be nearly as serious on the low dams as it would be on the high dam proposed under Alternate 1.

3. The dam on Cedar Creek, proposed under Alternate 2, will provide more complete flood protection to the City of Woodstock than the Cedar Creek Channel Improvement proposed under Alternate 1.

4. Under Alternate 2, the existing Canadian Pacific Railway spur line to the Woodstock Industrial Park, would remain, whereas under Alternate 1, it would have to be abandoned, thus depriving the existing industries in the Park of railroad service.

SECTION 13 - CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

1. The degree of flood protection is fully dependent upon the method of reservoir regulation.

2. The reservoir or reservoir and channel improvement as proposed under either Alternate, when combined with the proposed Woodstock Channel Improvement, will generally provide adequate flood protection for the City of Woodstock and for the quarry area between Beachville and Ingersoll if the reservoir(s) are operated in a manner similar to that described in this report.

3. The numerous advantages of Alternate 1 outweigh the slight advantage in flood crest reduction of Alternate 2, thus making Alternate 1 the more desirable alternate.

4. The proposed Woodstock Channel Improvement Extension will be effective in containing the design discharge only if the work proposed under Alternate 1 is constructed. A large enough channel to fully contain the Alternate 2 design discharge in this region would be drowned out by the head losses through the adjacent downstream bridges. However, the construction of the Woodstock Channel Improvement Extension would make the land in this area available for limited purposes such as parking lot or as a recreational area.

RECOMMENDATIONS

1. The various engineering works proposed under Alternate 1 should be constructed immediately, along with the proposed Woodstock Channel Improvement, to relieve flooding conditions in the City of Woodstock, to provide full flood protection for the quarry region between Beachville and Ingersoll, and to provide a necessary conservation pool in the region for low flow maintenance, irrigation and recreation.

2. A flood plain zoning program should be inaugurated for the low region adjacent to Cedar Creek between Mill Street and Finkle Street in the City of Woodstock to keep the existing open area free of new buildings and possible resultant future flood damage.

3. A reservoir regulation section should be set up to study and ascertain the most effective reservoir operating procedures for the dams proposed under Alternate 1 and for the other dams in the Thames River drainage basin in order to make full effective use of the storage volumes of these reservoir for both flood control and conservation purposes.

4. A program of erosion prevention should be established in the basin above the proposed reservoir site in order to reduce reservoir sedimentation and thus increase the life of the dam.

5. A study should be made of the inadequate section of the existing channel improvement between Mile 5.5, the start of the channel improvement, and the tributary joining the existing channel improvement at Mile 6.5, with the objective of reducing the peak flow in the channel in this region by the construction of a retarding dam or dams on the tributaries or by enlarging the channel itself.

6. Every effort should be made in the future to keep the flood plain of the Thames River in its existing condition so as to preserve natural flood plain storage. This goal can be aided by preventing land fill encroachments and by preventing the construction of any new channel improvements.

7. Every effort should be made to prevent new buildings, etc. from being constructed on the low lying portions of the flood plain where flooding and resultant flood damage will occur even with all proposed flood control works in place.

APPENDIX A - HYDROLOGY

GENERAL DESIGN PROCEDURE

The design procedure used in the hydrologic investigation of the drainage basin of the South Branch of the Thames River followed the general procedure used in the hydrologic investigation of most drainage basins. This procedure involved the sub-division of the South Branch of the Thames River into a series of reaches or sections, each of which subtends a sub-drainage basin or sub-area, the development of a unit hydrograph for each sub-drainage basin or sub-area, the computation of a design inflow hydrograph for each sub-drainage basin or sub-area by application of rainfall-excess (runoff) from the design rainfall or snowmelt to the unit hydrographs of each sub-drainage basin or sub-area, and finally, the combining and moving of the design inflow hydrograph downstream to the point of interest by the process of flood routing which determines the charges made to the design inflow hydrograph by the natural storage and artificial storage (dams) in the drainage basin. The theory and application of this design procedure to the South Branch of the Thames River is discussed in detail in this Appendix.

SELECTION OF RIVER REACHES

The selection of river reaches in the South Branch of the Thames River drainage basin and the resultant sub-drainage basins subtended by each reach was made in conformance with several general rules and criteria, applicable to small flat drainage basins, where there are no gauging stations to record flow, as described below:

1. Every point at which a hydrograph was desired, automatically set the head or foot of a river reach since the process of flood routing produces design hydrographs only at the head of each reach (upstream end of the sub-drainage basin) and at the foot of each reach (downstream end of the sub-drainage basin).

2. All points where major tributaries enter the main stem normally set the head or foot of a particular reach in order to facilitate the computation of design hydrographs and simplify flood routing procedures.

3. The river reaches are made as short and as uniform as possible so that accurate flood routing can be accomplished without an excessive amount of computation work. The necessity of these short routing reaches is discussed more fully in the section on flood routing.

Other criteria normally used to set the head or foot of river reaches in small drainage basins, such as at control points or at points where material changes in bottom slope or cross section occur, were not applicable to the portion of the Thames River drainage basin under study.

For purposes of defining and identifying the various sub-drainage basins subtended by the various river reaches, a stationline in miles was superimposed on the main stem of the South Branch of the Thames River and on all principal tributaries upstream from the intersection of the South Branch and the Middle Branch of the Thames River to the drainage divide as was mentioned in the main section of this report. This stationline is shown on Drawing No. 1

The various sub-drainage basins as was finally used in the hydrologic investigation of the South Branch of the Thames River drainage basin are listed in Tables 10 and 11 and are also outlined on Drawing No. 1.

TABLE 10

Sub-Drainage Basins - South Branch Thames River

No.	Description	River Reach From	To	Drainage Area (Sq. Miles)
1.	Ingersoll to Beachville	mile 6.4	mile 10.8	21.9
2.	Beachville to Beachville North	mile 10.8	mile 13.0	11.5
3.	Beachville North Governor's Road Br.	mile 13.0	mile 15.4	7.3
4.	Governor's Rd. Bridge to Woodstock Dam	mile 15.4	mile 17.3	10.2
5.	South Branch Thames River at Dam Site	above mile 17.3		93.5

TABLE 11

Sub-Drainage Basins - Cedar Creek

No.	Description	River Reach From	To	Drainage Area (Sq. Miles)
1.	South Branch Thames River to Cedar Creek Dam	mile 0.0	mile 2.8	3.6
2.	Cedar Creek at Dam Site	above mile 2.8		33.2

No effort was made to sub-divide the drainage basin above the dam site on either the South Branch of the Thames or on Cedar Creek, and in both cases unit hydrographs and design hydrographs were developed for the total drainage basins above the dam sites. Although there is a slight difference in location between the actual position of the dams on the South Branch of the Thames River near Woodstock, as proposed under the two Alternates, these locations were considered the same for purposes of the hydrologic study and investigation.

COMPUTATION OF A DESIGN RAINFALL

General:

In establishing a design rainfall (storm) for the design of flood control works, there are several general possible procedures.

One procedure is to use the records of actual rainfalls from severe storms that have occurred in the area under consideration, and from such records select some severe rainfall as being representative of the flood producing rainfalls in the area and actually use the plotted hyetograph or mass curve of such a rainfall or storm in design. A slight variation of this procedure, which is frequently used, is to construct a synthetic hyetograph or mass curve from the records of several storm or major rainfalls which have occurred as the "design" rainfall for the area.

Another procedure used extensively involves the preparation of rainfall intensity-frequency data by mathematical analysis of long term recording rainfall gauging station records and from such data establishing a synthetic rainfall hyetograph or mass curve. This procedure enables the engineer to assign an approximate frequency of occurrence (e.g. 100 year return period) to the synthetic design rainfall. However, all of these procedures require an extensive long term period of rainfall records, to be successfully used. Such records are not available in the Thames River Valley and vicinity as previously mentioned.

Fortunately, the drainage basin of the South Branch of the Thames River is located in the southern portion of Ontario which is surrounded on three sides by sections of the United States (New York, Ohio and Michigan) where many rainfall gauging stations with long periods of record are located. These rainfall gauging stations were considered close enough so that information as recorded by them could be transposed to and utilized in the Thames River Valley without significant error. Information concerning these adjacent gauging stations was obtained from published data of the United States Weather Bureau prepared for the use of the United States Soil Conservation Service on ungauged drainage basins similar to the Thames River basin[(1)]. The

(1) Rainfall Intensity - Frequency Regime, Weather Bureau Technical Paper No. 29, U.S. Government Printing Office, Washington 25, D.C.
Part 4 - Northeastern United States - May 1959
Part 5 - Great Lakes Region - February 1960

rainfall gauges at Detroit, Michigan and Buffalo, New York, were, for the purpose of this report, considered to be representative of conditions in the South Branch of the Thames River Valley.

These United States Weather Bureau publications enable the computation of a point rainfall for any particular frequency for any desired duration and provided charts and tables necessary for the reduction of the point rainfall for area and for computing seasonal or monthly rainfalls for a particular duration and frequency.

The 100 year frequency (return period) design rainfall was computed for the Thames River drainage basin, by use of these publications, and are described herein. As previously mentioned, a 24-hour rainfall duration was assumed to be the critical time period in the drainage basin for the design storm.

As a first step in computing the 24-hour duration 100-year frequency design rainfall, 24-hour-100 year frequency point rainfalls for the key rainfall gauging stations at Detroit, Michigan and Buffalo, New York, were computed and are listed as follows:

Location	Rainfall Depth (Inches)
Buffalo, New York	4.80
Detroit, Michigan	5.00

Since the drainage basin of the South Branch of the Thames River is closer to Buffalo, New York, the decision was made to give this gauge more weight than the Detroit, Michigan gauge. Based on this decision, a 24-hour duration, 100-year point rainfall of 4.85 inches was selected as the rainfall most representative of conditions in the drainage basin.

This computed 100 year frequency point rainfall valve was then reduced 7% as recommended in these United States Weather Bureau Publications, because of the large area of the drainage basin under consideration. This resulted in an average 24-hour duration, 100 year frequency design rainfall of 4.50 inches which was used as the Project Design Rainfall in this report.

Time Distribution of the Design Rainfall:

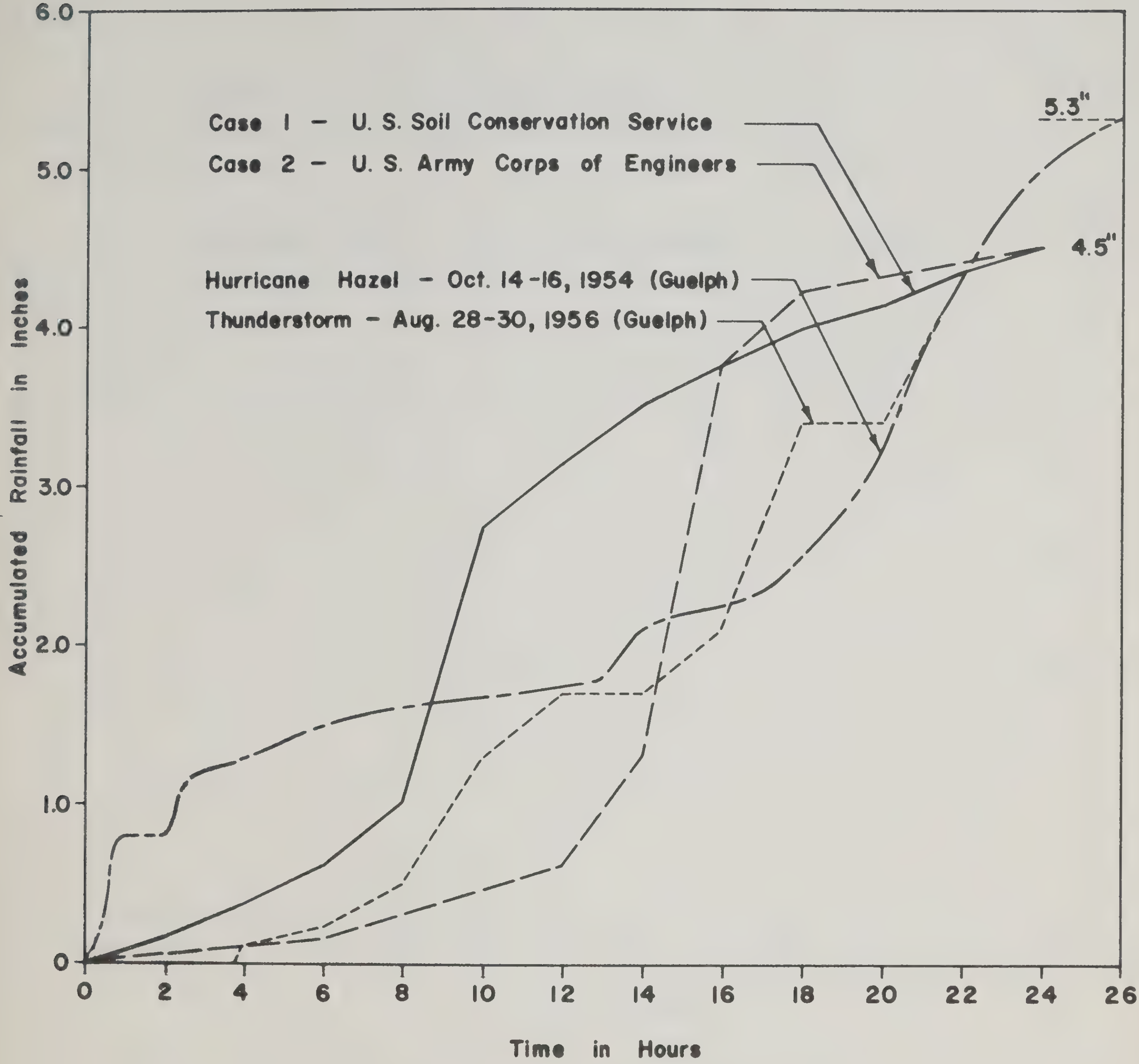
No information was provided in these United States Weather Bureau publications which permitted a breakdown or subdivision of the 24-hour, 100 year frequency design rainfall into one or two hour increments, as is required for the computation of a design flood resulting from such a rainfall. Therefore, in order to select a time-volume distribution (or subdivision) of the 24-hour, 100 year frequency project design rainfall, this quantity of rainfall (4.50 inches) was subdivided and mass curves plotted in accordance with the different standard methods used in design by both the United States Soil Conservation Service [(1)], and the United States Army Corps of Engineers [(2)], as shown on Figures 12 and 13. For purposes of comparison, the mass curves of two recent severe storms, specifically the storm of October 14-15, 1954 (Hurricane Hazel) and the thunderstorm of August 28-30, 1956, as recorded by rainfall gauging stations in the adjacent Grand River Basin, in the City of Guelph, Ontario, were also superimposed on these figures.

From the study of these plotted mass curves, it was immediately apparent that the more severe United States Army Corps of Engineer rainfall distribution more nearly matched the actual recorded mass curves and therefore was adopted for use in this report.

Hyetographs (graphical representations) of this average rainfall for the one and two hour periods are shown on Figures 22 through 24.

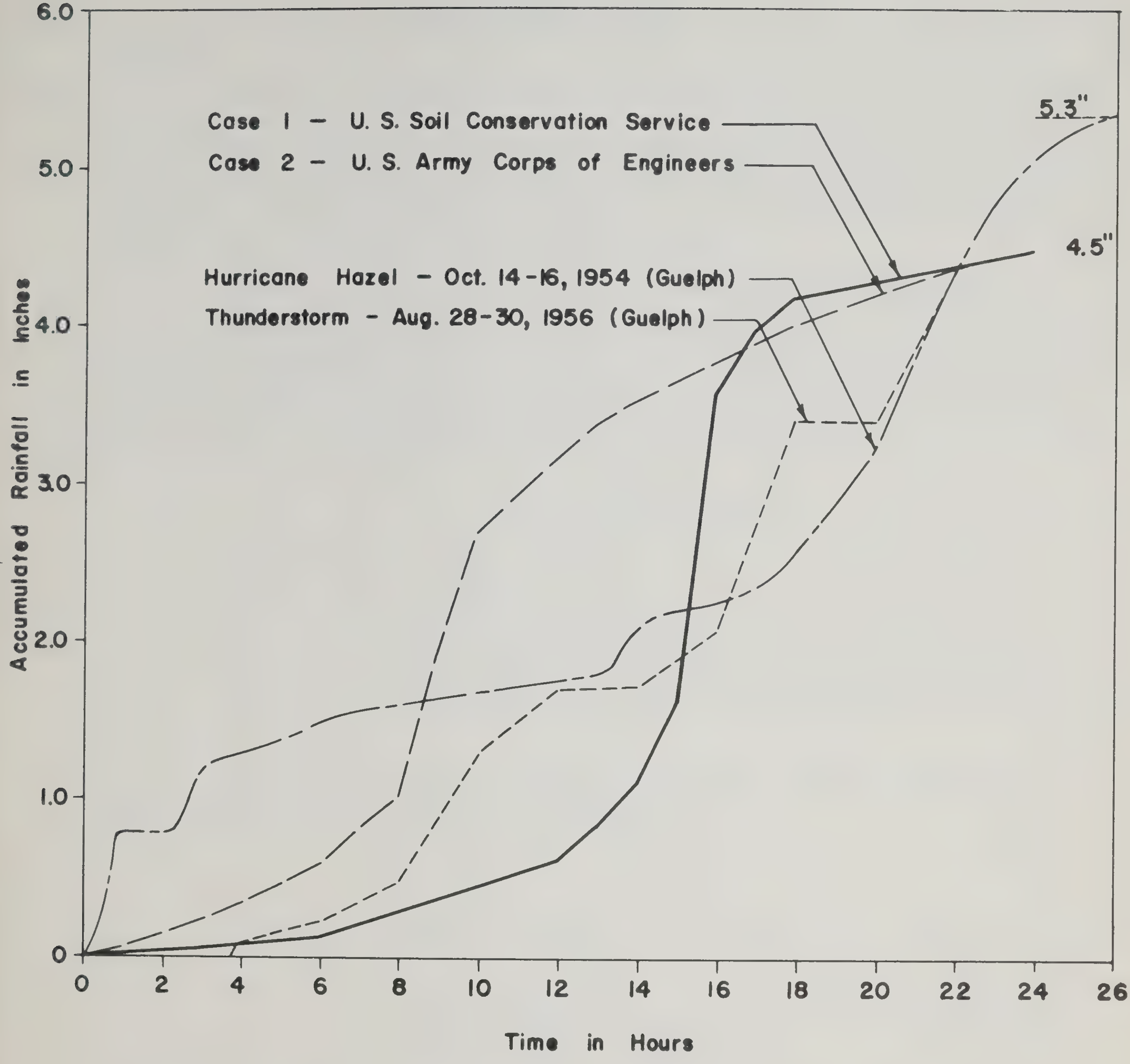
(1) "The Hydrology Guide for use in Watershed Planning" National Engineering Handbook, Section 4, Supp. A, Soil Conservation Service, Washington, D. C.

(2) "Standard Project Flood Determination" Civil Works Engineer, Bulletin 52-8 Corps of Engineers, U.S. Army, Washington, D.C.



MASS RAINFALL CURVES
100 YEAR FREQUENCY DESIGN RAINFALL
(TWO HOUR INCREMENTS)

FIGURE 12



MASS RAINFALL CURVES

100 YEAR FREQUENCY DESIGN RAINFALL

(ONE HOUR INCREMENTS)

FIGURE 13

UNIT HYDROGRAPHS

General:

With no major flood having occurred since the establishment of the existing recording stream gaging stations in the drainage basin of the Thames River, it was not possible to compute any actual unit hydrographs for the various drainage basins in the watershed. This fact required that synthetic unit hydrographs be computed for each of these sub-drainage basins of the South Branch of the Thames River.

Synthetic unit hydrographs for each of the sub-drainage basins were computed using a method developed by the Soil Conservation Service of the United States Department of Agriculture and used and recommended by both the Soil Conservation Service [1] and the United States Bureau of Reclamation [2]. The peak discharge (q_p) of the synthetic unit hydrograph as computed by this method is given by the equation:

$$q_p = \frac{484\,AQ}{T_p}, \quad \text{where}$$

q_p	=	peak discharge in cfs.
A	=	contributory drainage area in square miles
Q	=	total runoff in inches (= 1" for unit hydrograph)
T_p	=	time from beginning of use to peak of hydrograph in hours.

The time in hours from beginning of rise to the peak of the synthetic hydrograph is given by the equation:

$$T_p = D/2 + L, \quad \text{where}$$

D	=	unit storm duration (duration of the rainfall-excess) in hours.
L	=	basin lag, defined as the time interval from the centre of mass of the rainfall-excess to the peak of the hydrograph.

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(1) "The Hydrology Guide for Use in Watershed Planning". Soil Conservation Service, National Engineering Handbook, Section 4, Supplement A.

(2) "Design of Small Dams" Bureau of Reclamation, U.S.Dept. of the Interior, United States Government Printing Office, Washington, D.C. 1960.

Dimensionless unit hydrograph ratios developed by the Soil Conservation Service for use with the above equations giving the relationship between T_p and any other time (T) and q_p) and any other discharge Q are given in columns 1 and 2 of Table 16.

In the computation of the unit hydrographs for each of the various sub-basins in the watershed, the basin lag (L) was assumed equal to 0.6 of the time of concentration (T_c).

$$L == 0.6\ T_c$$

The time of concentration (T_c) is defined as the time of runoff to travel from the most remote point in the drainage basin to the point of interest. This equation is recommended for use in small drainage basins with a single principal watercourse, by both the United States Soil Conservation Service, and the Bureau of Reclamation, in the previously referenced publications.

Computation of the Time of Concentration.

As was previously mentioned, most of the river reaches for the drainage basins of the South Branch of the Thames River were selected so that the major tributary of the particular sub-drainage basin subtended by the river reach entered the main stem of the river at the head or foot of the reach. This procedure permitted the assumption to be made that the entire contributory area of the sub-drainage basin was concentrated either at the head or foot of the river reach. The entire runoff from the drainage area was considered to enter at one point instead of along the entire length of the reach, this facilitating the process of flood routing as will be described in a later section of this Appendix. Thus, in most cases, the computation of the time of concentration (T_c) involved merely the computation of travel time in the tributary from the basin divide to the main stem.

In the case of the major drainage basins above the dam site, the time of concentration was computed as the travel time from the basin divide to the individual dam site.

In both the cases of themajor stems of the river above the dam sites and for the sub-drainage basins, the time of concentration (T_c) was computed by substitution in the following equations developed by Snyder (1).

$$T_c = C_t L' 0.6 \quad \text{and}$$

$$L' = \frac{10\ L_n}{\sqrt{S}} \quad \text{where}$$

(1) "Synthetic Flood Frequency" by F.F.Snyder, Proceedings Paper 1908, Journal of the Hydraulics Division, Proceedings ASCE, October 1958.

C_t = coefficient dependant on type of drainage system.

L' = length of equivalent channel having the same time of concentration, but with a standard slope of 1% and a standard friction factor of 0.1

L = length of principal drainage channel

n = friction factor of channel (Manning's)

S = weighted slope of principal drainage channel in % determined as the mean height of the channel profile above the point of interest divided by one half the length.

A valve for $C_t = 2.0$ was recommended by Snyder for natural drainage basins and used to compute the time of concentration in each of the sub-drainage basins. From a thorough study and field inspection of the principal drainage channels in the Thames River drainage basin, a Manning's friction factor of $n = 0.050$ was considered to be the average valve in the drainage basin, for both summer and winter conditions, and was used to compute the time of concentration throughout the drainage basin. The length of the principal drainage channel of each sub-drainage basin was measured on topographic maps as the distance along the principal watercourse in the particular sub-drainage basin from the point of interest to the drainage divide. The weighted slope of each principal drainage channel was computed utilizing a procedure recommended by Golding and Low(1). This procedure involves the computation of the mean height of the channel profile by integrating the elevation distance curve and dividing by the channel length.

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(1) "Pyhsical Characteristics of Drainage Basins" by B.L.Golding & D.E.Low.
Proceedings paper 2409, Journal of the Hydraulics Division, Proceedings ASCE, March 1960.

TABLE 12

UNIT HYDROGRAPH CHARACTERISTICS

Sub-Drainage Basins - South Branch of the Thames River

Description	River Reach From	River Reach To	Drainage Area (sq. mi.)	Time of Concentration (T_c) (hours)	Lag Time (L) (hours)
Ingersoll to Beachville	mile 6.4	mile 10.8	34.8	6.22	3.7
Beachville to Beachville N	mile 10.8	mile 13.0	11.5	2.75	1.7
Beachville N to Gov. Rd. Bridge	mile 13.0	mile 15.4	7.3	2.70	1.7
Gov. Rd. Bridge to Woodstock Dam	mile 15.4	mile 17.3	10.2	4.18	2.5
South Branch of Thames River @ Dam Site	above mile 17.3		93.5	16.30	9.8

TABLE 13

UNIT HYDROGRAPH CHARACTERISTICS

Sub-Drainage Basins - Cedar Creek

Description	River Reach From	River Reach To	Drainage Area (sq. mi.)	Time of Concentration (T_c) (hours)	Lag Time (L) (hours)
South Branch Thames River to Cedar Creek Dam	mile 0.0	mile 2.8	3.6	2.02	1.2
Cedar Creek @ Dam Site	above mile 2.8		33.2	7.18	4.3

TABLE 14

PHYSICAL CHARACTERISTICS
Sub-Drainage Basins - South Branch of the Thames River

Description	River Reach From	River Reach To	Drainage Area	Length of Principal Drainage Channel(L) (mi.)	Mean Height of Channel Profile (ft)	Weighted Slope of Principal Drainage Channel (S) (%)
Ingersoll to Beachville	mile 6.4	mile 10.8	34.8	8.4	88.1	0.398
Beachville to Beachville N	mile 10.8	mile 13.0	11.5	3.7	113.8	1.17
Beachville N to Gov. Rd. Bridge	mile 13.0	mile 15.4	7.3	3.4	112.0	1.27
Gov. Rd Bridge to Woodstock Dam	mile 15.4	mile 17.3	10.2	5.9	113.0	0.732
South Branch of Thames River @ Dam Site	above mile 17.3		93.5	26.0	106.0	0.1546

TABLE 15

PHYSICAL CHARACTERISTICS
Sub-Drainage Basins - Cedar Creek

Description	River Reach From	River Reach To	Drainage Area (Sq. mi)	Length of Principal Drainage Channel(L) (mi)	Mean Height of Channel Profile (ft)	Weighted Slope of Principal Drainage Channel (S) (%)
South Branch Thames River to Cedar Creek Dam	mile 0.0	mile 2.8	3.6	2.8	10.9	0.148
Cedar Creek Dam Site	above mile 2.8		33.2	7.6	41.2	0.204

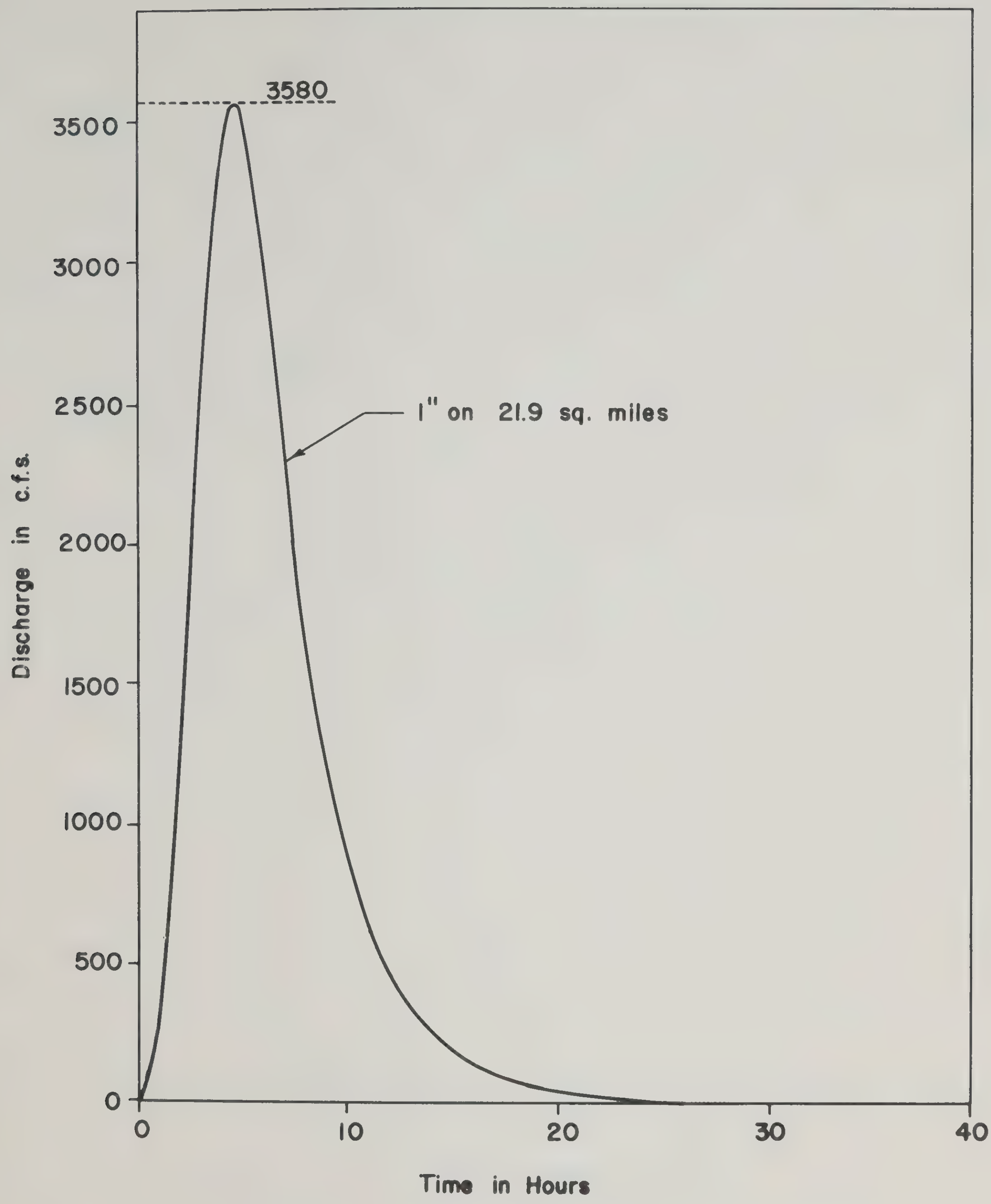
TABLE 16

UNIT HYDROGRAPH OF SOUTH BRANCH OF THAMES RIVER
COMPUTATION OF 2 HOUR SYNTHETIC UNIT HYDROGRAPH

SOUTH BRANCH OF THAMES RIVER AT DAM SITE

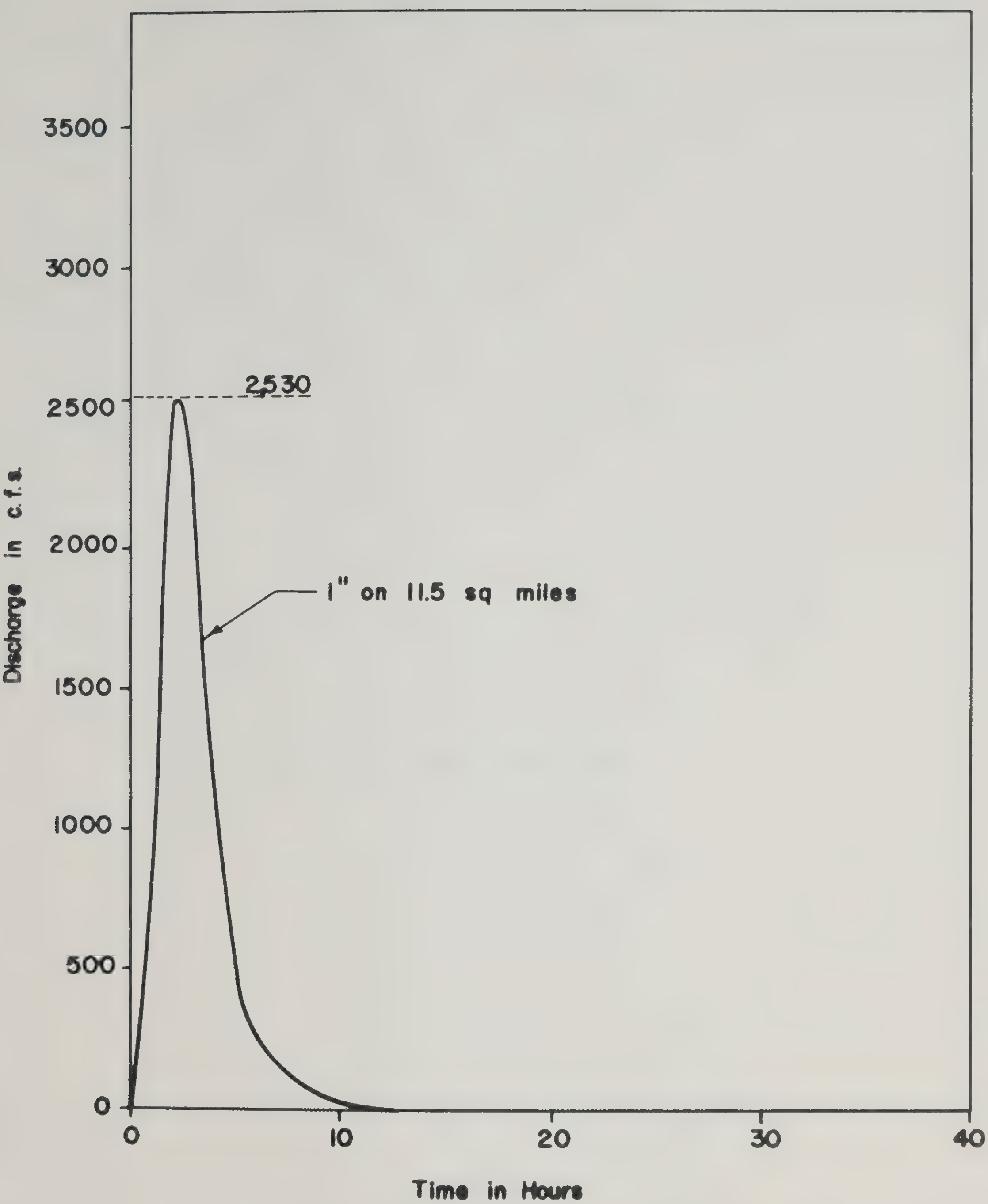
Computation of Unit Hydrograph				Clock Hour Readings	
Time Ratio (T/T_p)	Discharge Ratio q/qp	Time (T) (hours)	Discharge (q) (cfs)	Time (hours)	Discharge (cfs)
0	0.	0.	0	0	0
0.1	0.015	1.08	63	2	310
0.2	0.075	2.16	315	4	1005
0.3	0.16	3.24	672	6	2135
0.4	0.28	4.32	1176	8	3470
0.5	0.43	5.40	1807	10	4100
0.6	0.60	6.48	2520	12	4100
0.7	0.77	7.56	3235	14	3530
0.8	0.89	8.64	3740	16	2775
0.9	0.97	9.72	4070	18	2170
1.0	1.00	10.80	4200	20	1650
1.1	0.98	11.88	4115	22	1260
1.2	0.92	12.96	3865	24	970
1.3	0.84	14.04	3530	26	730
1.4	0.75	15.12	3150	28	550
1.5	0.65	16.20	2730	30	420
1.6	0.57	17.28	2395	32	325
1.8	0.43	19.44	1807	34	250
2.0	0.32	21.60	1345	36	190
2.2	0.24	23.76	1009	38	140
2.4	0.18	25.92	756	40	105
2.6	0.13	28.08	546	42	80
2.8	0.098	30.24	412	44	70
3.0	0.075	32.40	315	46	50
3.5	0.036	37.84	151	48	40
4.0	0.018	43.20	76	50	30
4.5	0.009	48.60	38	52	20
5.0	0.004	54.00	17	54	15
				56	0

This procedure for computing the time of concentration was not used on the Cedar Creek sub-drainage basin below the dam site of the South Branch of the Thames River to Cedar Creek Dam (mile 0.0 to mile 2.8) due to it's small size. In this particular case, the time of concentration was considered to be the travel time from the head to the foot of the reach. The computed physical characteristics of the various sub-drainage basins in the South Branch of the Thames River watershed are shown on Tables 12 and 13. The unit hydrograph characteristics, of such as peak discharge (Q_p) and time of rise to peak (T_p) for the various sub-drainage basins in the South Branch of the Thames River Watershed are shown on Tables 14 and 15. The computation of the 2 hour synthetic unit hydrograph for the South Branch of the Thames River at the Dam Site is shown on Table 16. The unit hydrographs for the various sub-drainage basins are shown on Figures 14 to 20.



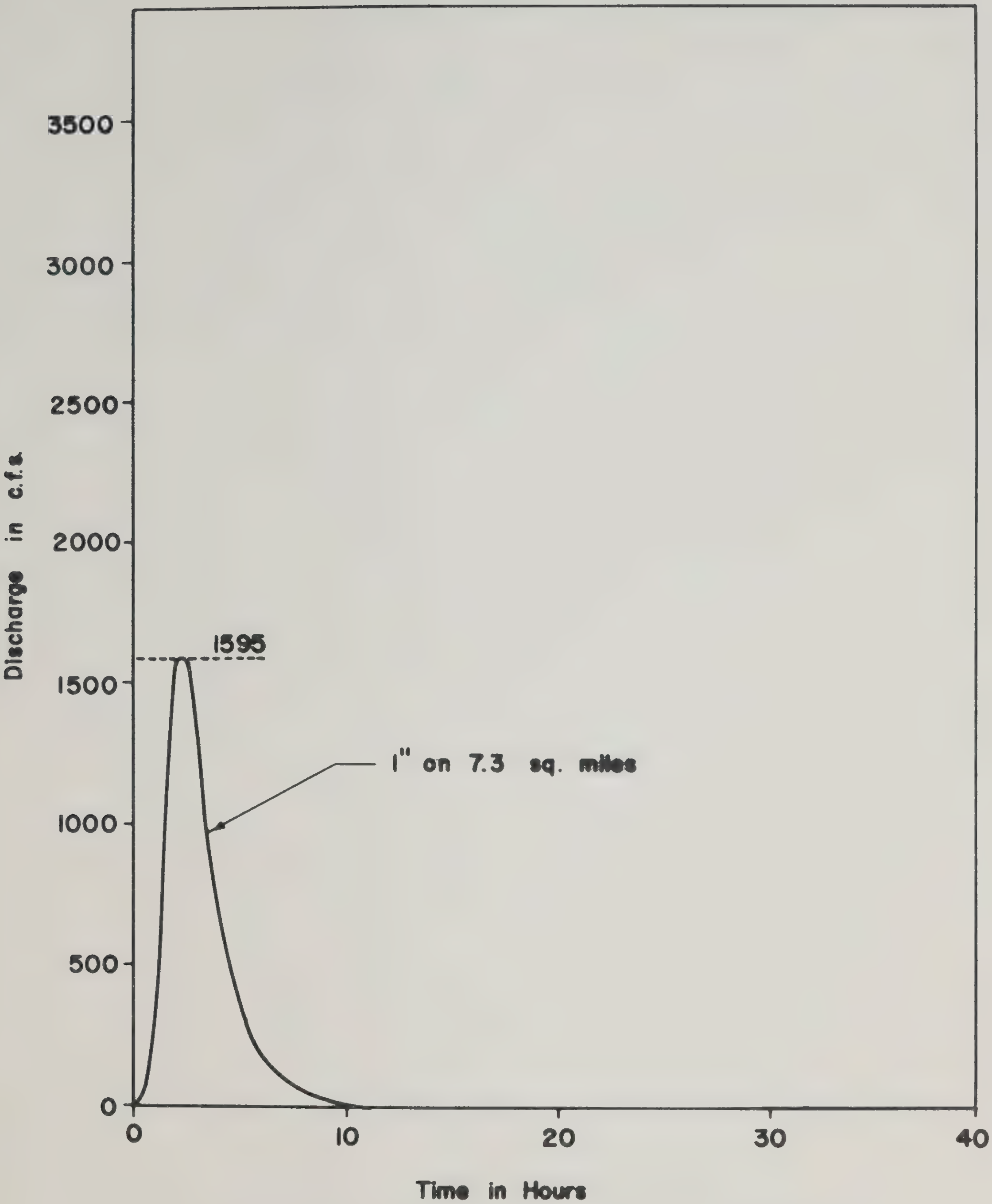
2 HOUR SYNTHETIC UNIT HYDROGRAPH
SOUTH BRANCH THAMES RIVER
MILE 6.4 TO MILE 10.8
(INGERSOLL TO BEACHVILLE)

FIGURE 14



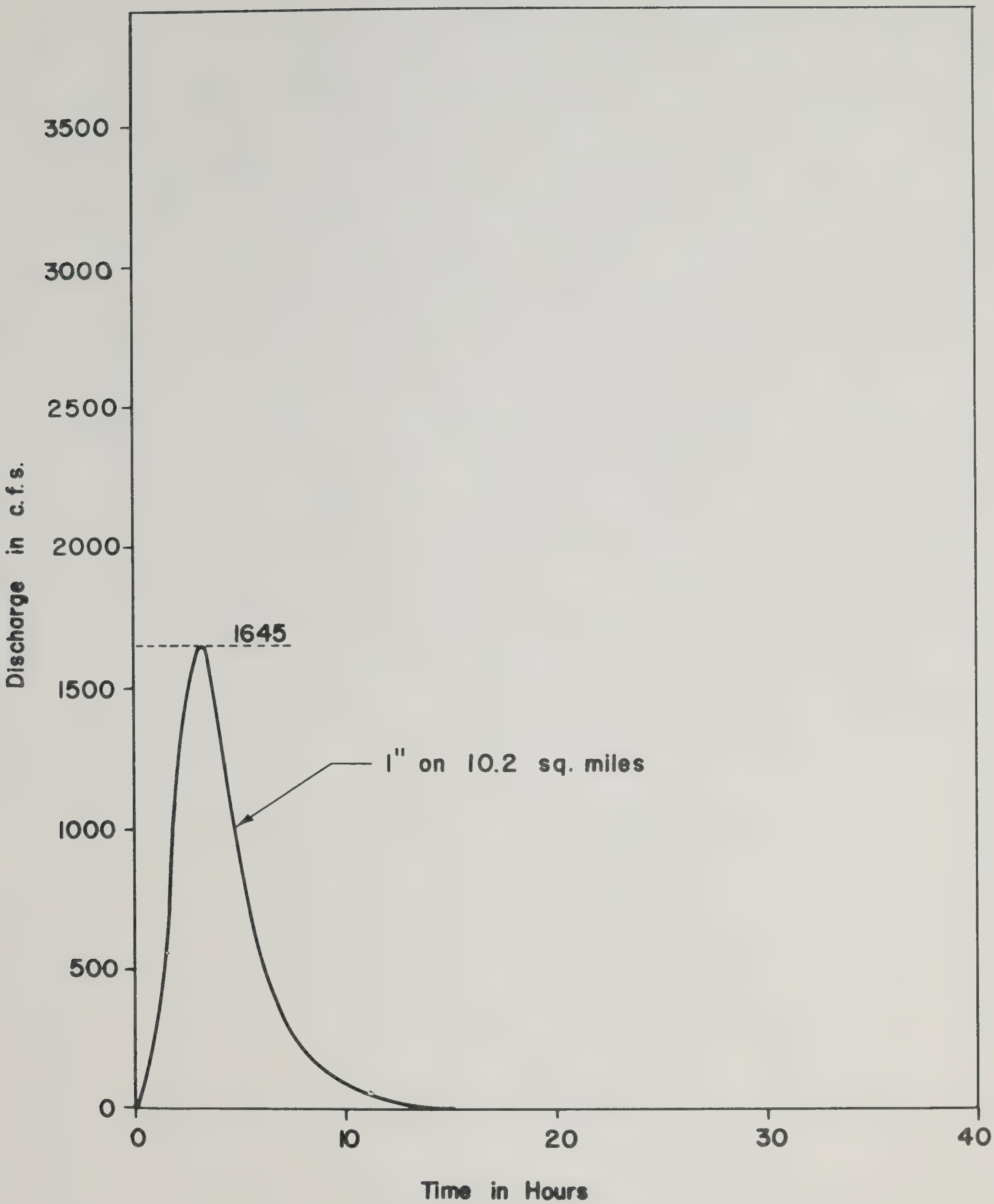
I HOUR SYNTHETIC UNIT HYDROGRAPH
SOUTH BRANCH THAMES RIVER
MILE 10.8 TO MILE 13.0
(BEACHVILLE TO BEACHVILLE NORTH)

FIGURE 15



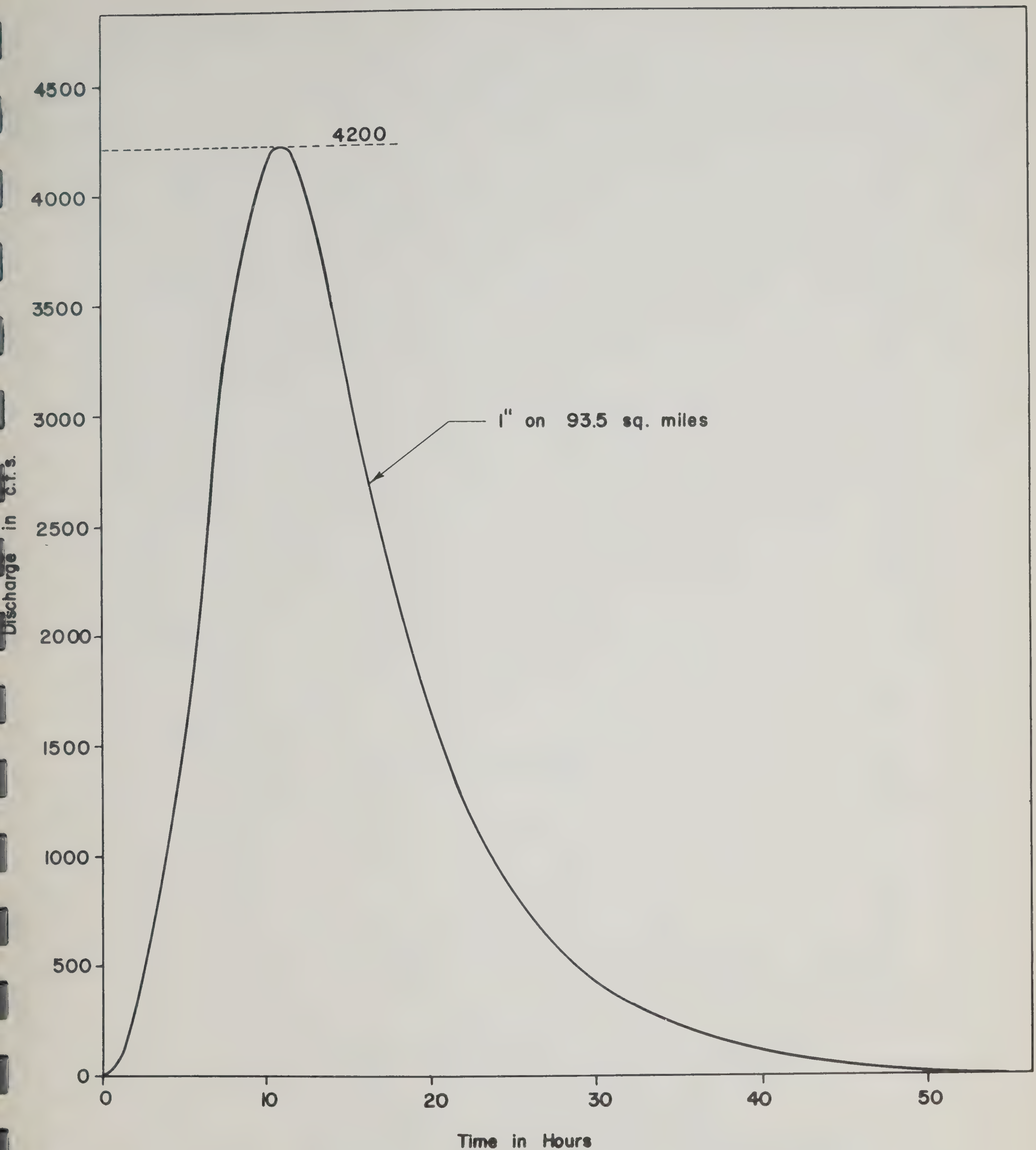
I HOUR SYNTHETIC UNIT HYDROGRAPH
SOUTH BRANCH THAMES RIVER
MILE 13.0 TO MILE 15.4
(BEACHVILLE NORTH TO GOVERNOR'S ROAD BRIDGE)

FIGURE 16



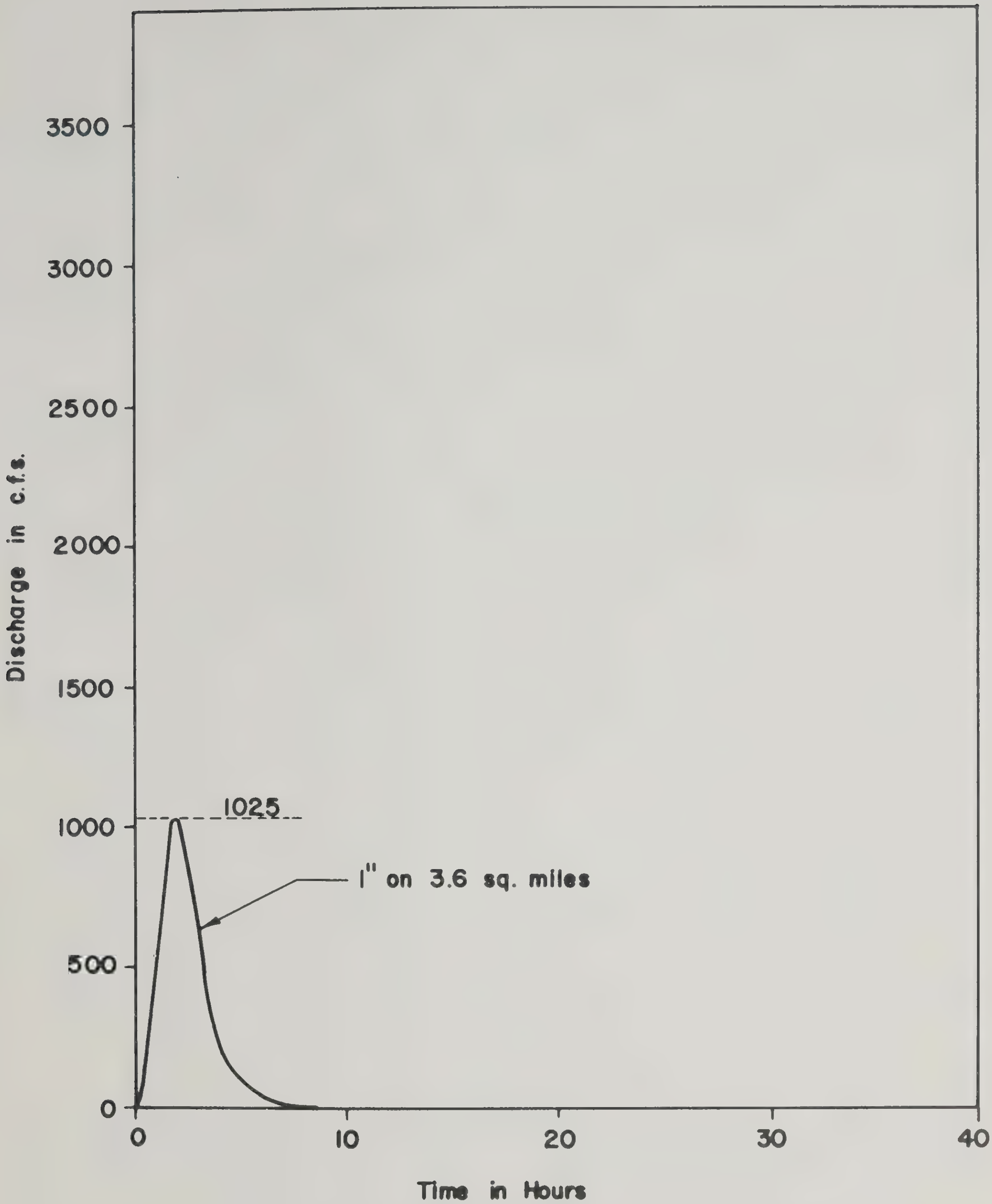
I HOUR SYNTHETIC UNIT HYDROGRAPH
SOUTH BRANCH THAMES RIVER
MILE 15.4 TO MILE 17.3
(GOVERNOR'S ROAD BRIDGE TO WOODSTOCK DAM)

FIGURE 17



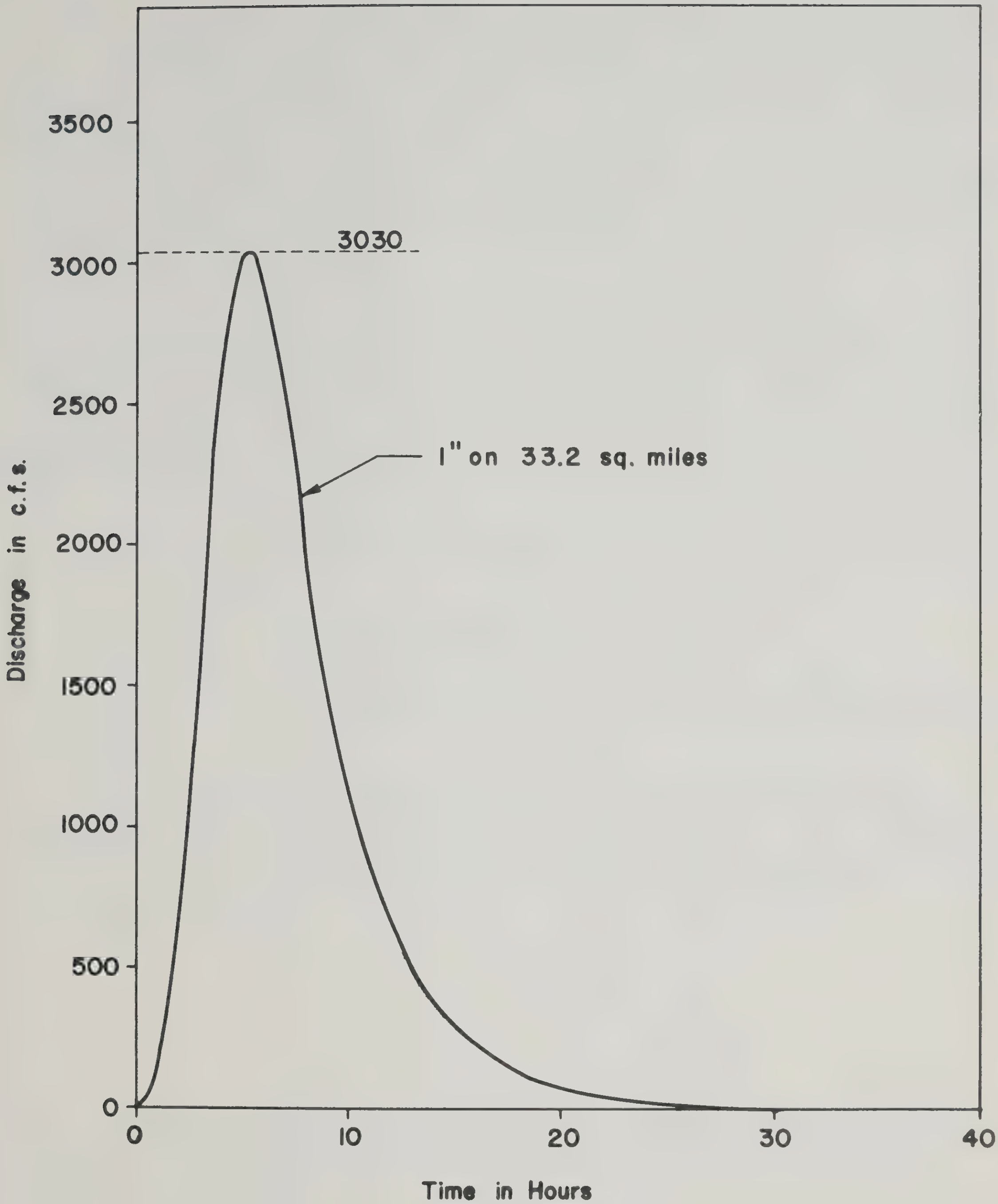
2 HOUR SYNTHETIC UNIT HYDROGRAPH
SOUTH BRANCH THAMES RIVER AT DAM SITE
MILE 17.3

FIGURE 18



I HOUR SYNTHETIC UNIT HYDROGRAPH
CEDAR CREEK
MILE 0.0 TO MILE 2.8
(DAM SITE TO SOUTH BRANCH THAMES RIVER)

FIGURE 19



2 HOUR SYNTHETIC UNIT HYDROGRAPH
CEDAR CREEK AT DAM SITE
MILE 2.8

FIGURE 20

COMPUTATION OF DIRECT RUNOFF (RAINFALL-EXCESS)

Normally, the actual computation of direct runoff (rainfall-infiltration) into ground, depends upon previously computed infiltration indices for the drainage basin under consideration. However, due to the fact that computed infiltration indices, which are dependent upon gauging stations, were not available in the Thames River drainage basin, an alternate method developed by the United States Soil Conservation Service[1] was used.

This alternate method involves the use of an equation for the computation of the direct volume of runoff (Q) in inches from the storm rainfall (P) in inches, based on antecedent rainfall conditions, the hydrologic soil group, the land use or cover, and the treatment or farming practice, which are lumped into a term referred to as the Hydrologic Soil Cover Complex. This equation is given as follows:

$$Q = \frac{(P-0.2S)^2}{(P-0.8S)}, \quad \text{where}$$

Q = direct runoff in inches
P = storm rainfall in inches
S = maximum potential difference between P & Q in inches, at the time of the storm's beginning.

The solution of this equation is greatly simplified by use of a graph developed by the United States Soil Conservation Service and reproduced in Figure 21. The curve numbers in this graph depend upon the antecedent rainfall and the Hydrologic Soil Cover Complex, which are described below.

(1) "The Hydrology Guide for use in Watershed Planning" - National Engineering Handbook, Section 4, Supplement A, Soil Conservation Service, Washington D. C.

In computing direct runoff by this method, the Soil Conservation Service set up three moisture conditions for the ground in a particular drainage basin depending upon the amount of rainfall (antecedent rainfall) that fell previously. These conditions are listed as follows:

Condition 1 - A condition of drainage basin soils where the soils are dry but not to the wilting point.

Condition 2 - The average case, where the five day antecedent rainfall varies from 1.5 to 2.0 inches.

Condition 3 - When heavy rainfall or light rainfall and low temperatures have occurred during the five days prior to the given storm.

Antecedent Rainfall Condition 3 was assumed as the soil moisture condition for the computation of the Design Flood Hydrographs due to the actual wet conditions which seem to be characteristic of the region prior to the occurrence of a major storm. The antecedent conditions, assumed prior to the occurrence of the maximum probable flood (spillway hydrograph) are discussed in the section of this Appendix titled "Computation of the Maximum Probable Flood".

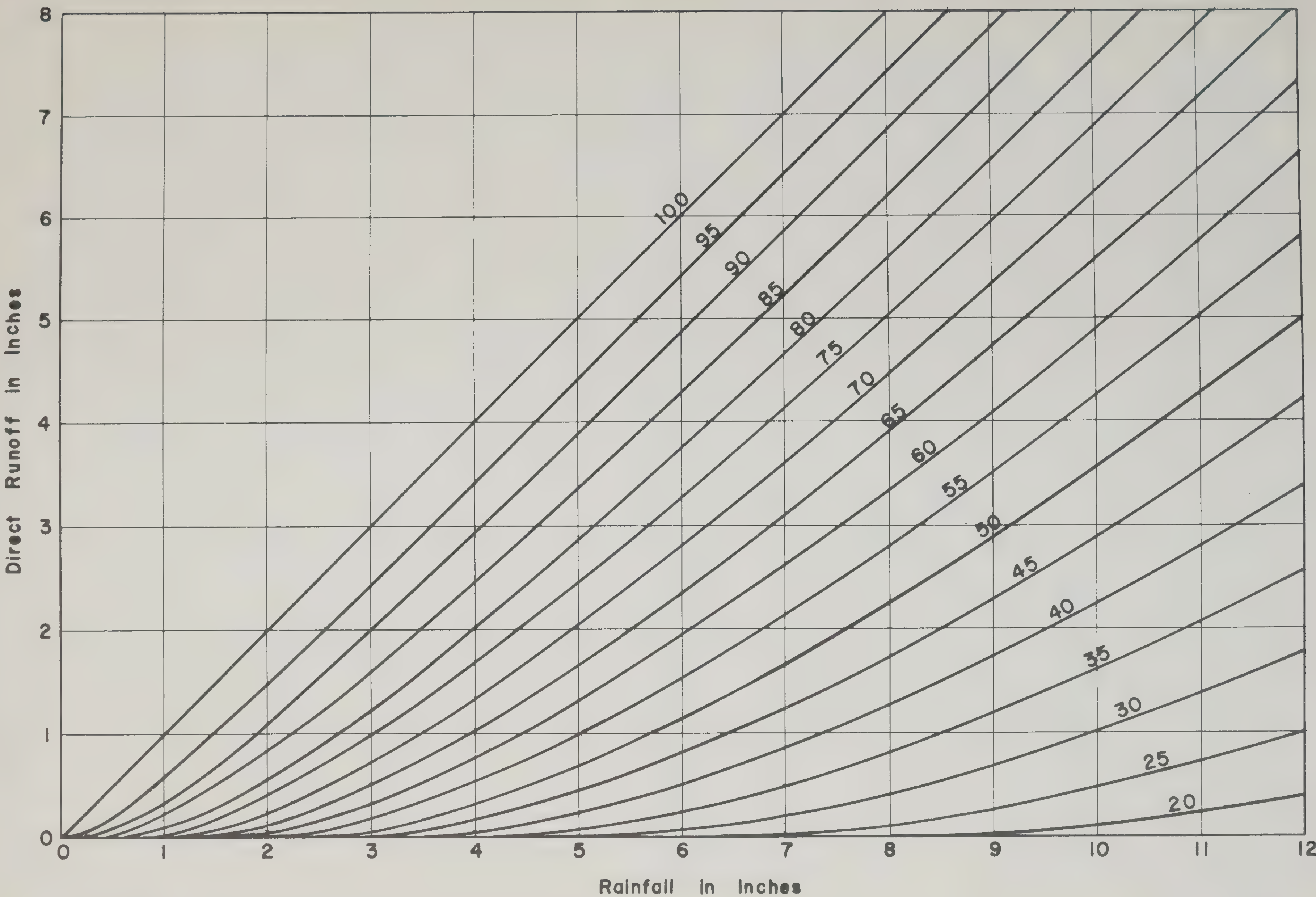
In the determination of the hydrologic soil cover complex, the SCS classifies soils into four major groups, based upon the intake of water at the end of long duration storms occurring after prior wetting and opportunity of swelling and without the protective effects of vegetation. These groups are listed as follows:

Group A - Lowest runoff potential includes deep sands with very little silt and clay, also deep, rapidly permeable loess.

Group B - Mostly sand soils, less deep than Group A, but the group as a whole has above average infiltration after thorough wetting.

Group C - Comprises shallow soils and soils containing considerable clay loam and colloid, though less than those of Group D. This group has below-average infiltration after pre-saturation.

Group D - Highest runoff potential includes mostly clays of highest swelling percent, but the group also includes some shallow soils with nearly impermeable subhorizons near the surface.



RAINFALL — DIRECT RUNOFF CURVES

U. S. SOIL CONSERVATION SERVICE

FIGURE 21

The various types of soils in the Thames River drainage basin as obtained from the Agricultural Soils Report for the County of Oxford were classified into these four major groups by the Engineering and Soil Department Faculties of the Ontario Agricultural College at Guelph, Ontario. These classifications are given in Table 17 below.

Table 17

Series	Type	Hydrologic Soil Group
Tavistock	silt loam	C
Guelph	loam	B
Perth	clay loam	C
Guelph-Honeywell	loam, silt loam	B
Fox	loamy sand	A
Huron	clay or silt loam	D
Embro	silt loam	C
London'	loam	B
Parkhill	loam	C
Brisbane	sandy loam	B
Granby	sandy loam	B

Based on weighting the area of each type of soil, each of the sub-drainage basins of the South Branch of the Thames River and Cedar Creek was given a general hydrologic soil grouping as shown in Tables 18 and 19.

Table 18

Assigned Hydrologic Soil Grouping

Sub-drainage Basins - South Branch of Thames River

Description	River Reach From	River Reach To	Hydrologic Soil Grouping
Ingersoll to Beachville	mile 6.4	mile 10.8	B
Beachville to Beachville N.	mile 10.8	mile 13.0	B
Beachville N. to Governor's Rd. Bridge	mile 13.0	mile 15.0	B
Governor's Rd. Bridge to Woodstock Dam	mile 15.4	mile 17.3	B
South Branch of Thames River at Dam Site	above mile 17.3		C

Table 19

Assigned Hydrologic Soil Grouping

Sub-drainage Basins - Cedar Creek

Description	River Reach From	River Reach To	Hydrologic Soil Grouping
South Branch Thames River to Cedar Creek Dam	mile 0.0	mile 2.8	B
Cedar Creek at Dam Site	above mile 2.8		C

TABLE 20 RUNOFF CURVE NUMBERS FOR HYDROLOGIC SOIL-COVER COMPLEXES, FOR WATERSHED CONDITION 2.

Land use or cover	Treatment or practice	Hydrologic condition	Hydrologic Soil Group A	B	C	D
Fallow	Straight row		77	86	91	94
Row crops	"	Poor	72	81	88	91
	"	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	"	Good	65	75	82	86
	" and terraced	Poor	66	74	80	82
	" " "	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	" and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight row	Poor	66	77	85	89
	" "	Good	58	72	81	89
	Contoured	Poor	64	75	83	85
	"	Good	55	69	78	83
	" and terraced	Poor	63	73	80	83
	" and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	"	Fair	25	59	75	83
	"	Good	6	35	70	79
Meadow (permanent)		Good	30	58	71	78
Woods (farm woodlots)		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		- - -	59	74	82	86
Roads (dirt) 2/		- - -	72	82	87	89
(hard surface) 2/		- - -	74	84	90	92

1/ close-drilled or broadcast
2/ including right-of-way.

Table 21

Average Land Use or Cover in Oxford County

Category	Acres	Percent
Fallow (winter wheat)	13,900	3.1
Row crops	48,620	10.7
Small grains	103,680	22.9
Close-seeded legumes (hay)	76,300	16.8
Pasture (or range)	92,000	20.3
Meadow (permanent)	36,000	7.9
Woods (farm woodlots)	32,000	7.0
Farmsteads	6,000	1.3
Roads (including R.O.W.)	14,500	3.2
Swamps(& wasteland)	25,000	5.5
Urban areas	6,000	1.3
	454,000	100.0

Used in this report and shown on Table 20 are runoff curve numbers developed by the United States Soil Conservation Service for each of the hydrologic soil groups for Antecedent Moisture Condition 2 for each land use or cover, and treatment. Normally, the United States Soil Conservation Service uses this table to compute an average runoff curve number for each of the various sub-drainage basins in a particular watershed by the process of weighting the various curve numbers by multiplying the area or portion of each sub-drainage basin, which has a particular listed type land use or cover, and treatment or practice, summing up the resultant weighted numbers, and then dividing by the drainage area of the watershed to obtain an average runoff curve number for the basin. Since sufficient information to break down each of the selected sub-drainage basins of the South Branch of the Thames River was not available, and since the preparation of such a breakdown was impractical for a report of this type, the decision was made to assume that each sub-drainage basin had the same average land use or cover, and treatment or practice as for the whole of Oxford County. The land use or cover breakdown for Oxford County is shown on Table 21. Also according to the engineer of the Ontario Department of Agriculture, almost all the crops and grain of Oxford County are planted in straight rows and their hydrologic condition is varying from good to poor.

Using table 20 for Antecedent Moisture Condition 2, and based upon a weighted County-wide breakdown of land use or cover, the runoff curve number for each of the two general hydrologic soil groups classification in the County was computed and is listed on Table 22 below.

TABLE 22

RUNOFF CURVES FOR ANTECEDENT MOISTURE CONDITION 2

Hydrologic Soil Group	Runoff Curve No.
B	75
C	82

A method for conversion of the runoff curve number in Table 22 which are given with reference to Antecedent Moisture Condition 2, to an equivalent runoff curve number for Antecedent Moisture Conditions 1 and 3, as developed by the United States Soil Conservation Service, are given in Table 24, on the next page. Using this table, the equivalent runoff curve number for Antecedent Moisture Condition 3 for each of the Hydrologic Soil Groups which was used in the computation of the Project Design Flood are listed in Table 23 below.

TABLE 23

RUNOFF CURVES FOR ANTECEDENT MOISTURE CONDITION 3

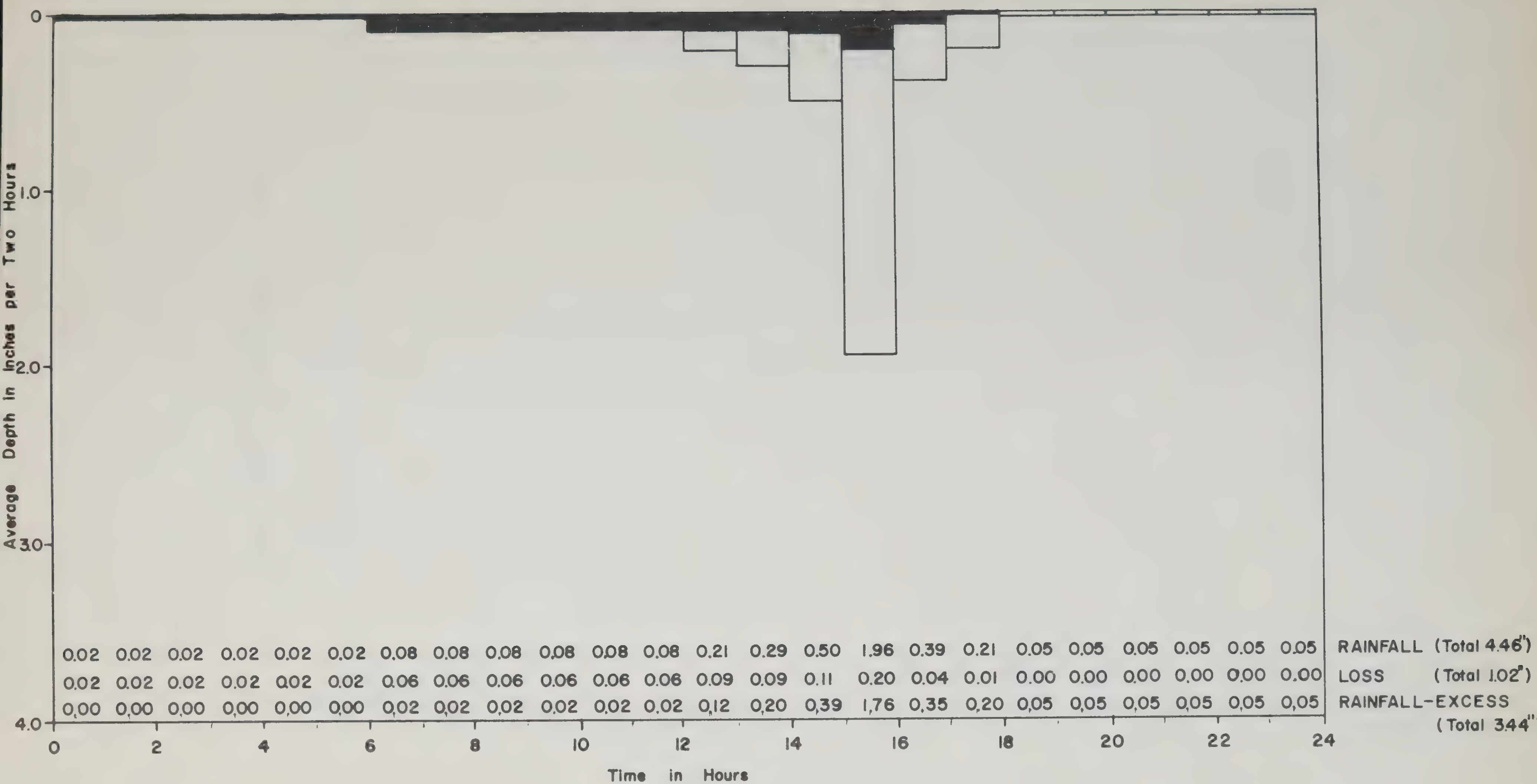
Hydrologic Soil Group	Runoff Curve No.
B	91
C	95

Using Runoff Curve No 95 as indicated on Figure 21 for Soil Type C, the direct runoff (rainfall-excess) was computed for the two hour incremental design rainfall since only two-hour unit hydrographs were computed for the sub-drainage basins with Type C soils. Using Runoff Curve No. 91 for Soil Type B, the direct runoff (rainfall excess) was computed for both the one hour and two hour incremental design rainfalls as both one hour and two hour unit hydrographs were developed for sub-drinage basins with Type B soils. Hyetographs (graphical representation) of average rainfall and rainfall-excess rates or volumes over the drainage basin, during successive units of time for each of the above conditions for the design rainfall, are shown on Figures 22 through 24.

TABLE 24

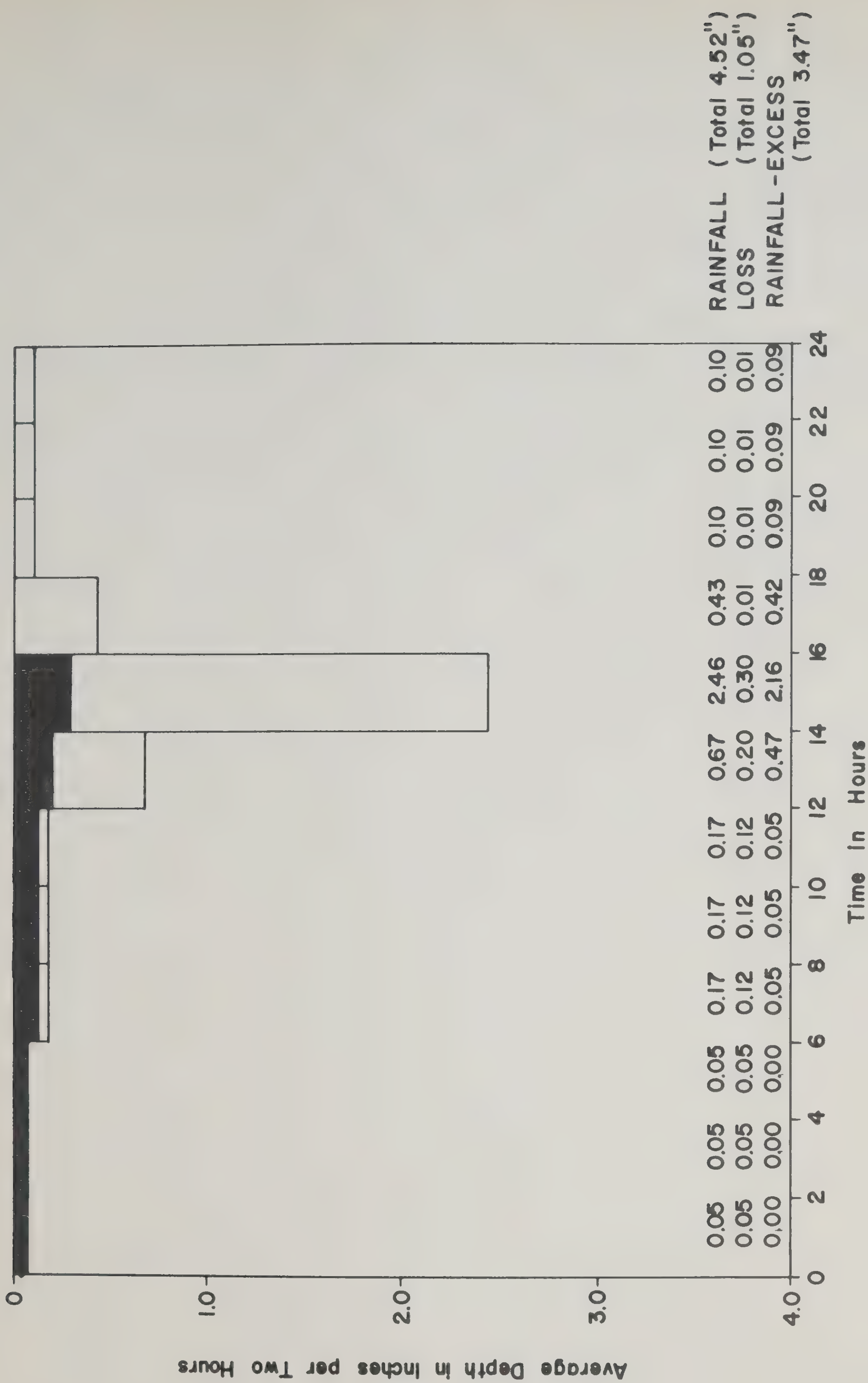
RUNOFF CURVE NUMBER CONVERSIONS
ANTECEDENT MOISTURE CONDITION 2 TO ANTECEDENT MOISTURE CONDITION 1 and 3.

Curve number for Condition 2	Corresponding Curve Numbers For Condition 1	Condition 3
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0



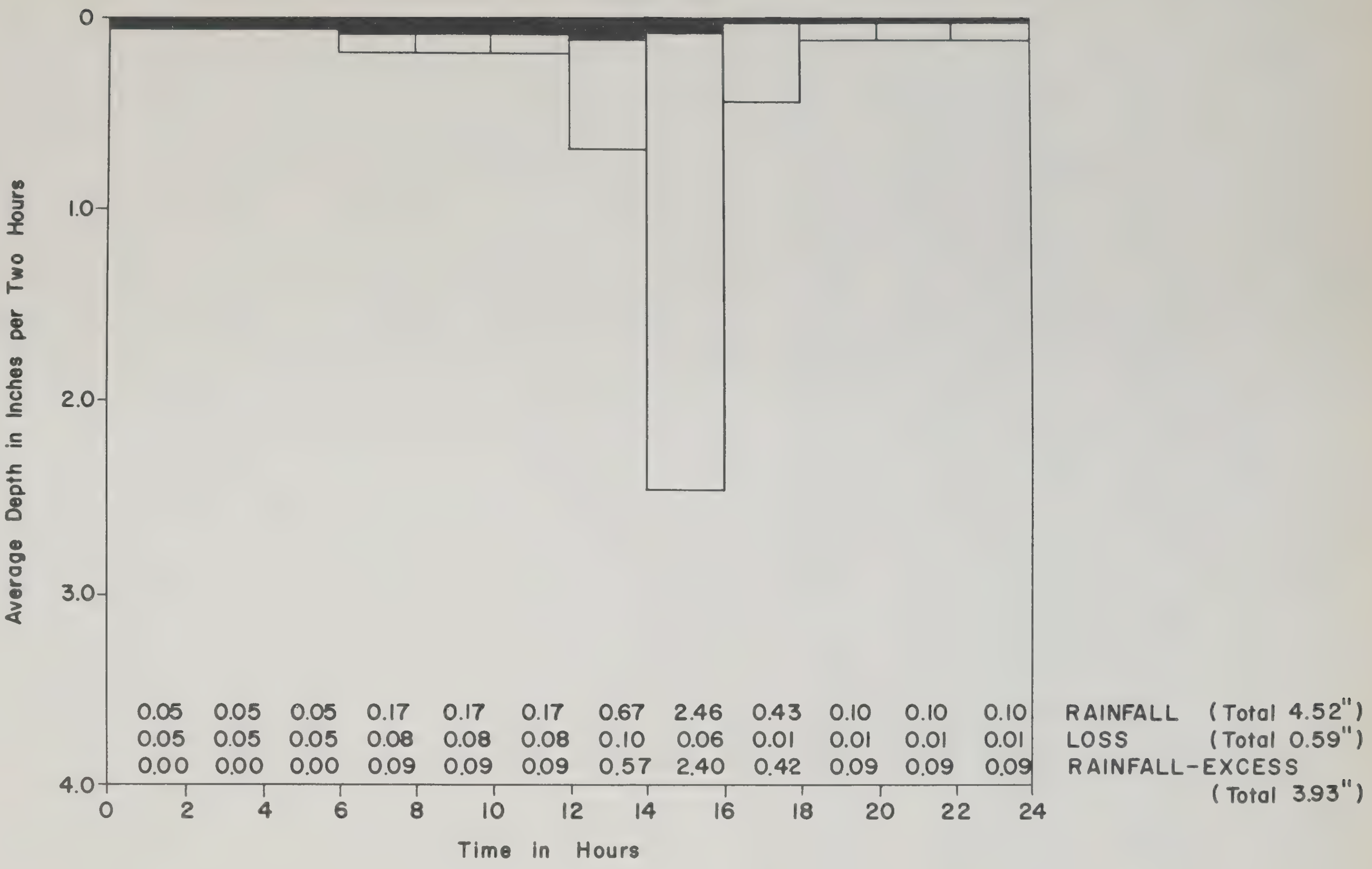
HYETOGRAPH OF DESIGN STORM
(ONE HOUR INCREMENTS)
SOIL TYPE B

FIGURE 22



HYETOGRAPH OF DESIGN STORM
(TWO HOUR INCREMENTS)
SOIL TYPE B

FIGURE 23



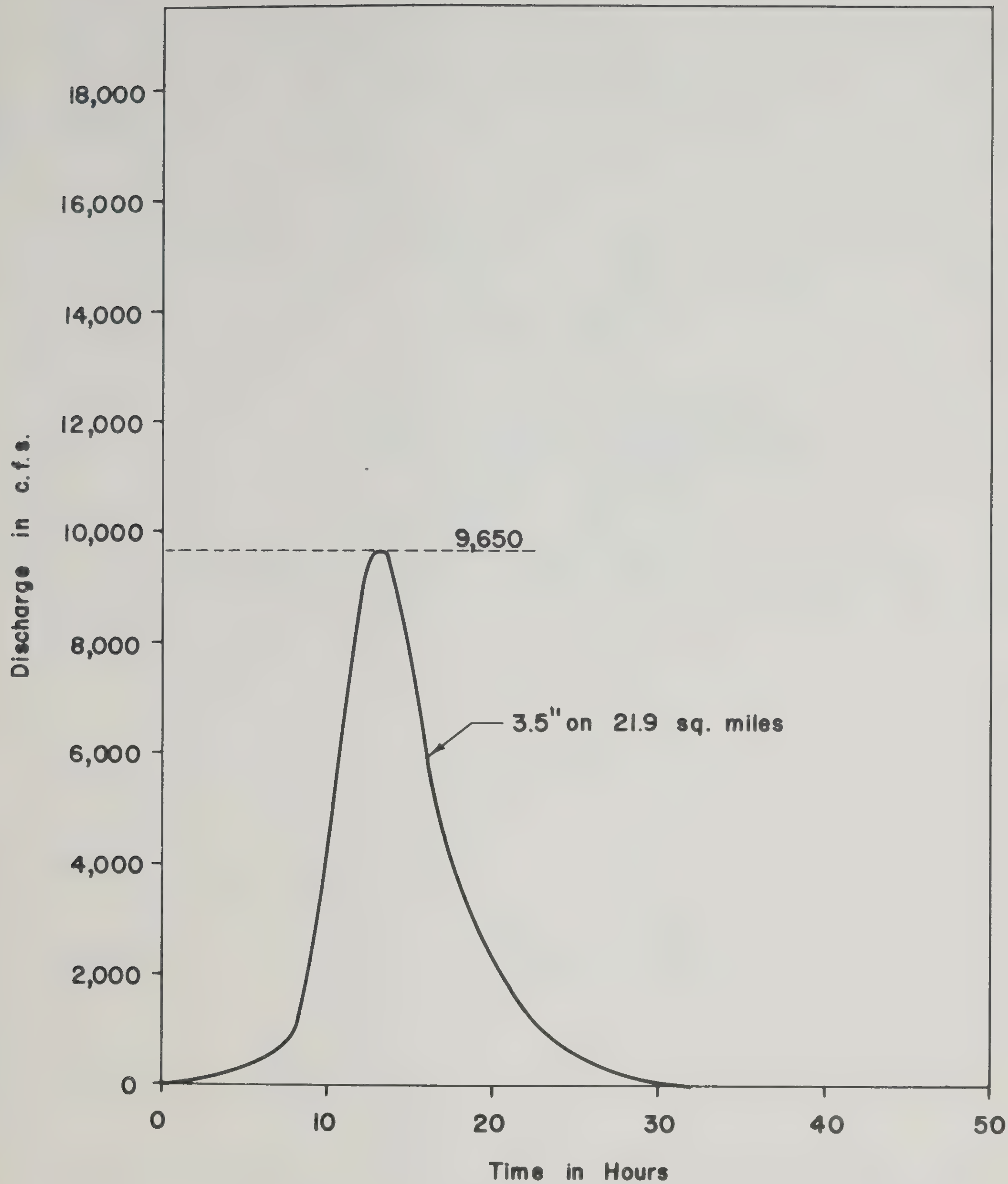
HYETOGRAPH OF DESIGN STORM
(TWO HOUR INCREMENTS)
SOIL TYPE C

FIGURE 24

COMPUTATION OF DESIGN FLOOD HYDROGRAPHS

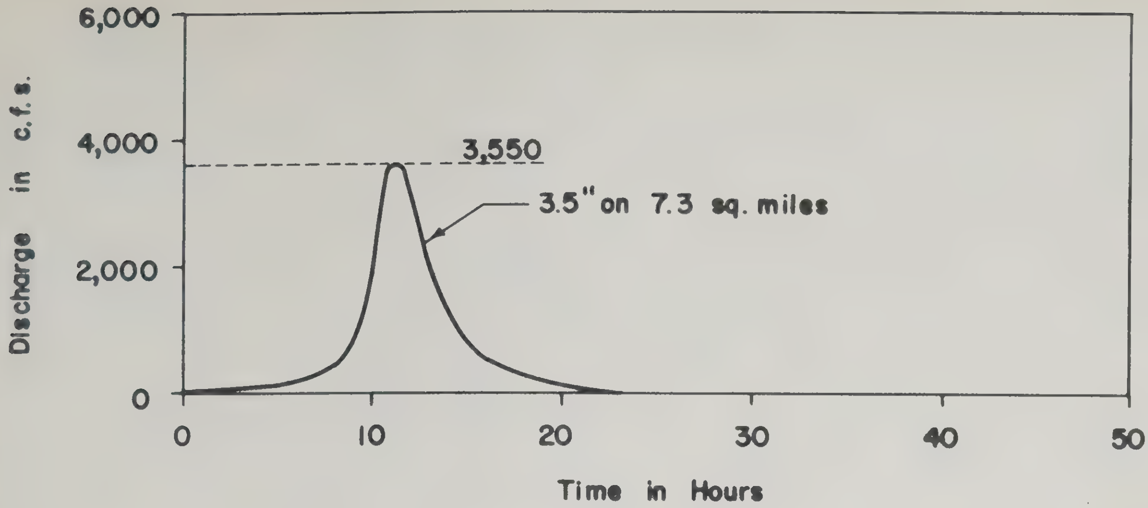
The computation of Design Flood Hydrographs for each of the principal drainage basins above the dam site and for the sub-drainage basins below the dam sites was made by multiplying the ordinates of the particular unit hydrograph by successive rainfall excess amounts (depending upon the soil type and duration of the unit hydrograph) and summing up the several partial hydrographs obtained in the usual manner.

The Design Flood Hydrographs for each of the sub-drainage basins are shown on Figures 29 through 31.



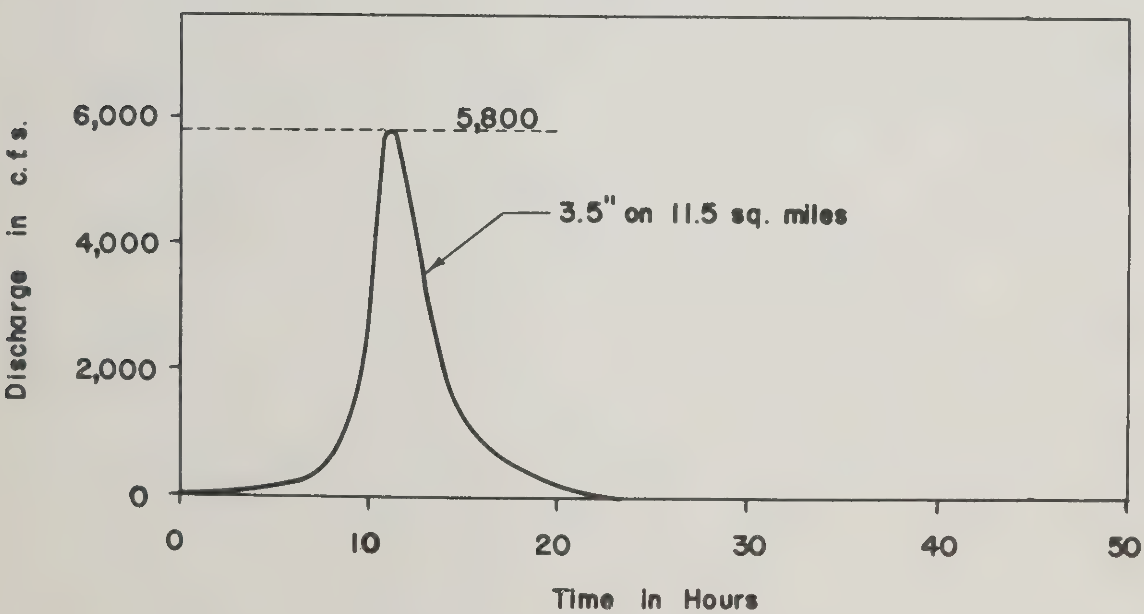
DESIGN FLOOD HYDROGRAPH
SOUTH BRANCH THAMES RIVER
LOCAL INFLOW
MILE 6.4 TO MILE 10.8
(INGERSOLL TO BEACHVILLE)

FIGURE 25



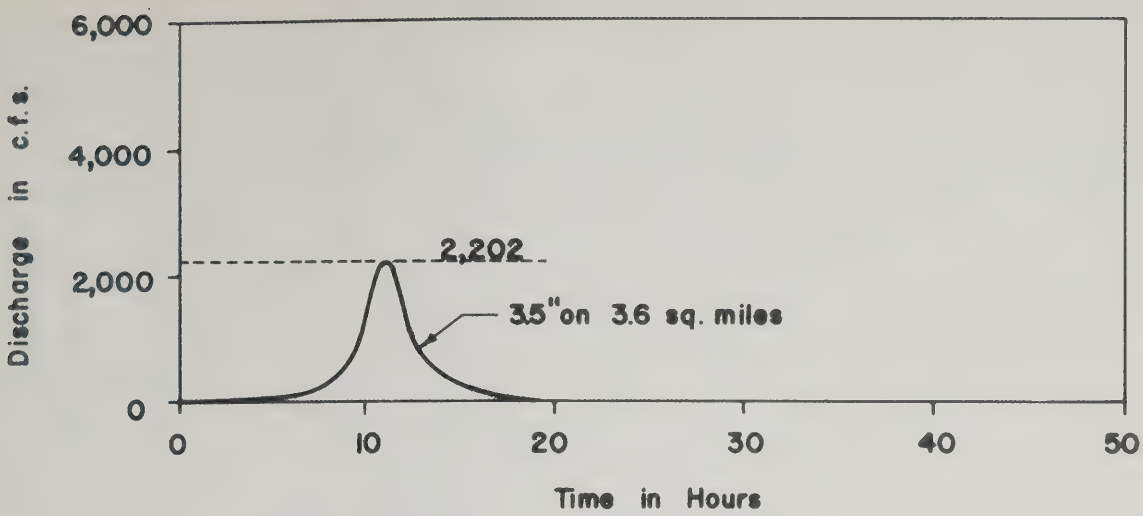
DESIGN FLOOD HYDROGRAPH
SOUTH BRANCH THAMES RIVER
LOCAL INFLOW
MILE 13.0 TO MILE 15.4
(BEACHVILLE NORTH TO GOVERNOR'S ROAD BRIDGE)

FIGURE 27



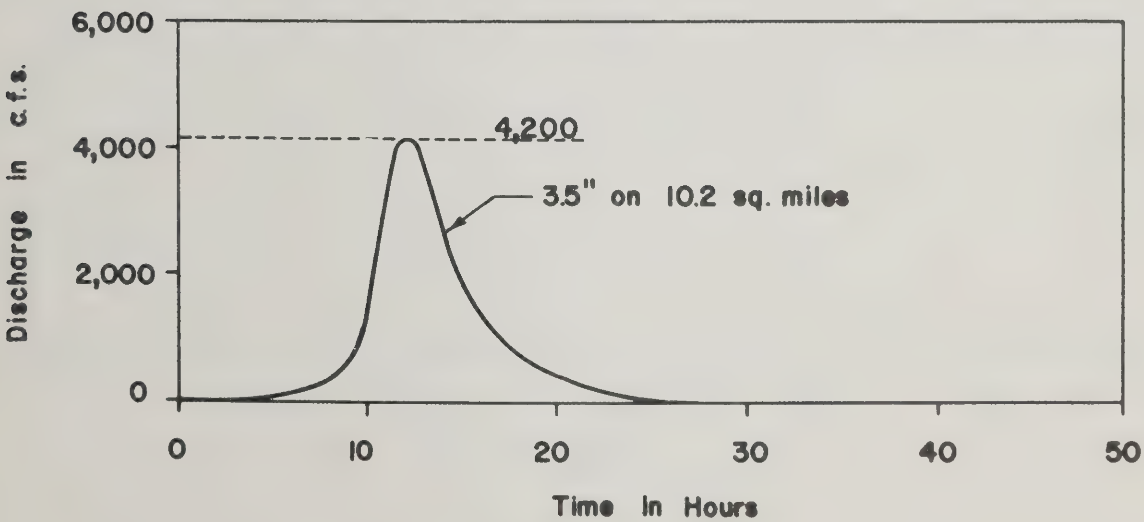
DESIGN FLOOD HYDROGRAPH
SOUTH BRANCH THAMES RIVER
LOCAL INFLOW
MILE 10.8 TO MILE 13.0
(BEACHVILLE TO BEACHVILLE NORTH)

FIGURE 26



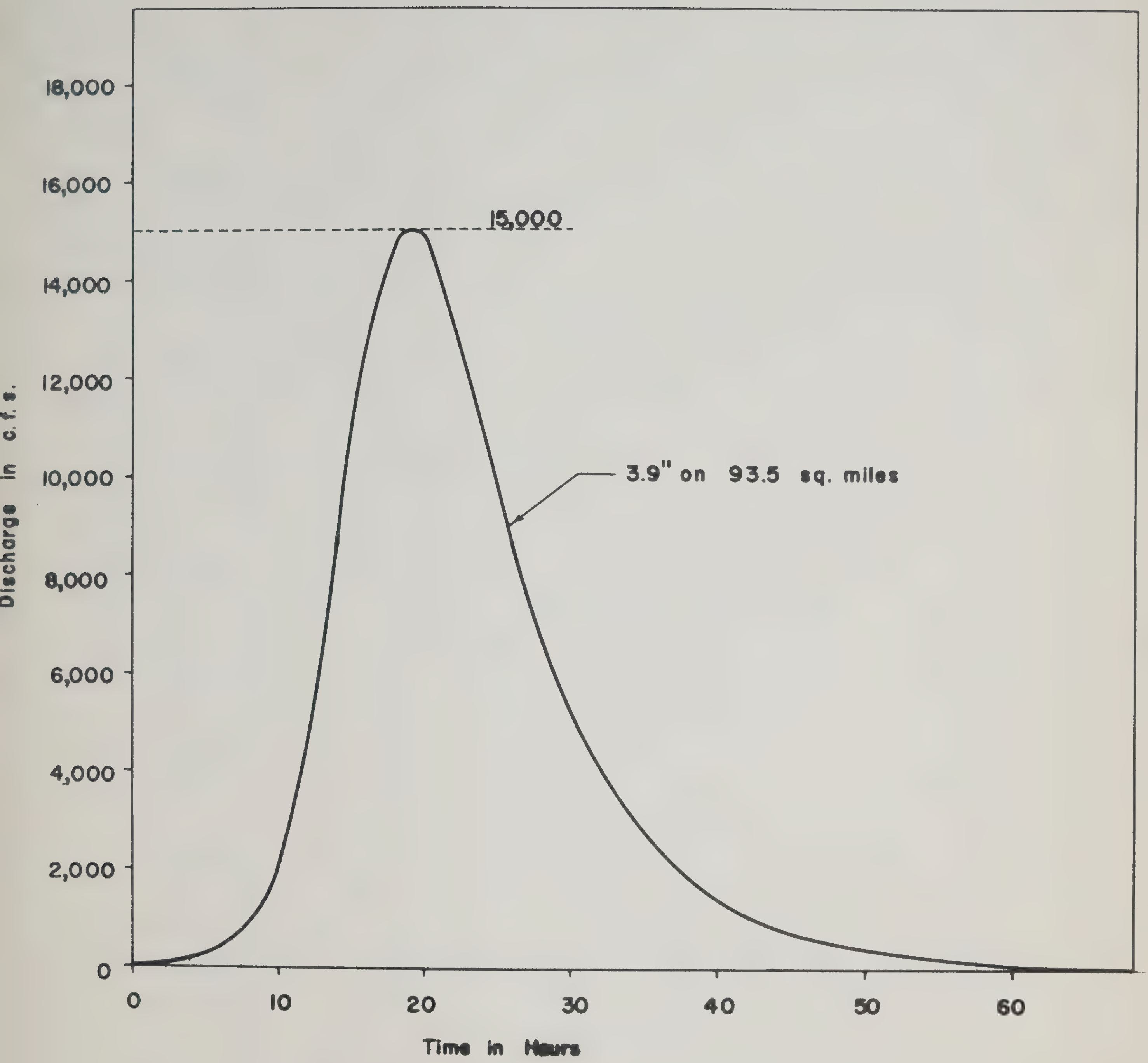
DESIGN FLOOD HYDROGRAPH
CEDAR CREEK
LOCAL INFLOW
MILE 0.0 TO MILE 2.8
(SOUTH BRANCH THAMES RIVER TO DAM SITE)

FIGURE 28



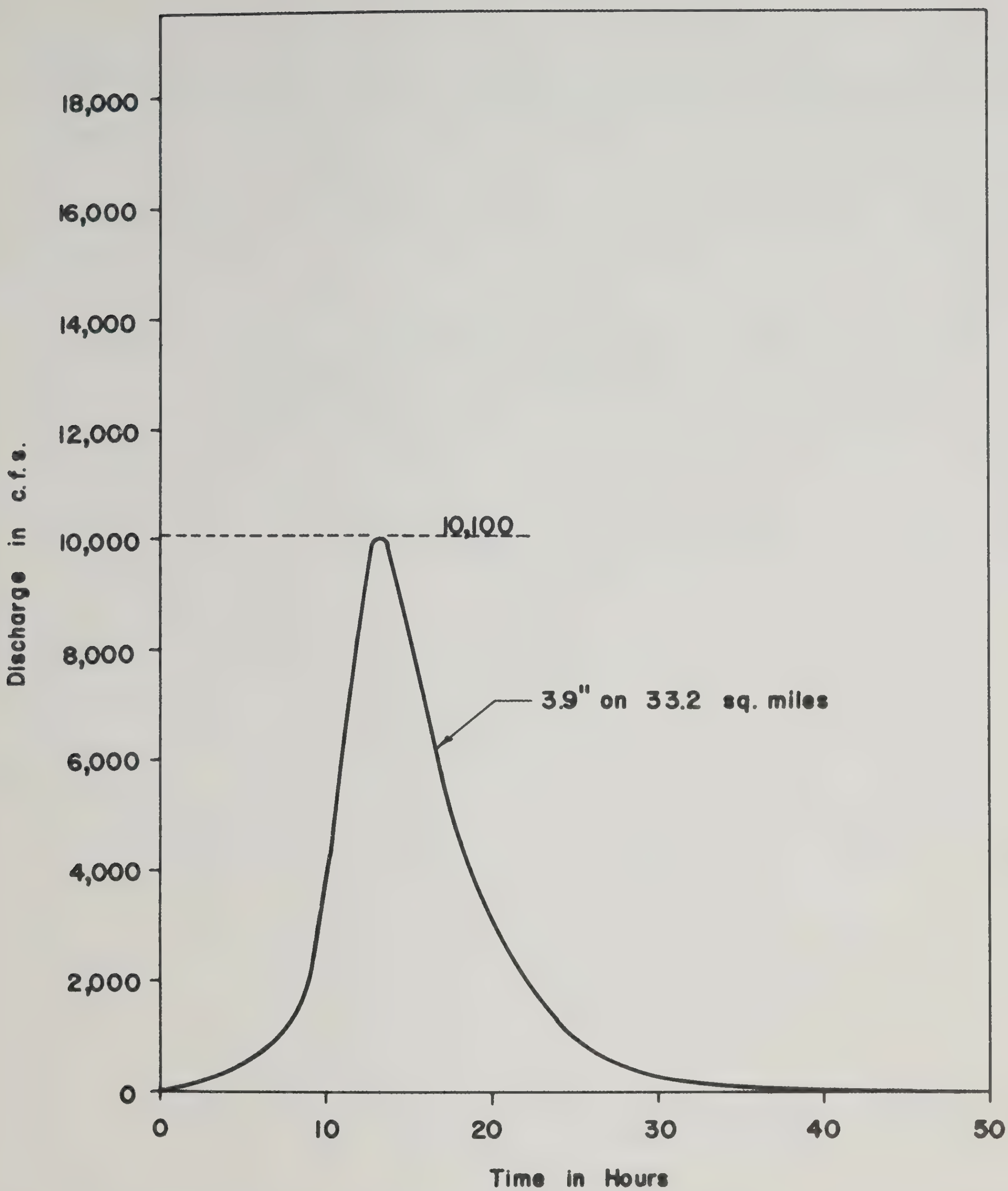
DESIGN FLOOD HYDROGRAPH
SOUTH BRANCH THAMES RIVER
LOCAL INFLOW
MILE 15.4 TO MILE 17.3
(GOVERNOR'S ROAD BRIDGE TO WOODSTOCK DAM)

FIGURE 29



DESIGN FLOOD HYDROGRAPH
SOUTH BRANCH THAMES RIVER AT DAM SITE
MILE 17.3

FIGURE 30



DESIGN FLOOD HYDROGRAPH
CEDAR CREEK AT DAM SITE
MILE 2.8

FIGURE 31

FLOOD ROUTING

General:

All flood routing computations in this report were based on the following general storage (flood routing) equation for a particular river reach:

$$I - O = \Delta S$$

where

I = volume of inflow into the reach for a given time interval
O = volume of outflow out of the reach for that time interval
ΔS = change in the volume of storage for that time interval

This equation is also written as

$$(I_1 + I_2)\ \Delta t - (O_1 + O_2)\ \Delta t = S_2 - S_1 = \Delta S$$

where

Δt = time interval $(t = t_2 - t_1)$

I_1 = inflow at time 1 (rate)

I_2 = inflow at time 2

O_1 = outflow at time 1

O_2 = outflow at time 2

S_1 = storage at time 1 (volume)

S_2 = storage at time 2

ΔS = change in volume of storage for the time interval

For purposes of this report the above equation was rewritten in the following manner.

$$\frac{I_1 + I_2}{2} + \frac{S_1}{\Delta t} - \frac{O_1}{2} = \frac{S_2}{\Delta t} + \frac{O_2}{2}$$

Normally the routing of a flood through a river reach requires a straam gauging station with fairly long periods of record be located at each end of the reach to record inflow to, and outflow from the reach. Generally, this enables the establishment of a relationship between outflow and / or inflow and storage in the reach, thus enabling the solution of the preceeding equations. However, since there were no gauging stations with long periods of record in the drainage basin of the South Branch of the Thames River or a sufficient number of stations to establish such relationship at the required points of interest, an alternate procedure was used.

This alternate procedure required the use of short river reaches, which, when considered along with the extreme flatness of the channel of the river under consideration, enabled the assumption that the wedge storage in each of the river reaches was negligible in comparison with the prismatic storage. This, in turn, enabled the assumption that the storage in the reach was directly proportional to and a direct function of the outflow.

In order to obtain the relationship between outflow and storage, backwater curves, as described in Appendix B, were first computed at various discharges along the entire length of the rivers, between Ingersoll and the Dam Sites, to obtain the relationship through the various reaches, between discharge and elevation, for both the proposed and existing conditions. The storage was then computed for each reach, at each discharge, by weighting the lengths between sections within the reach and summing up the volumes between sections, which volumes were based upon the areas at each section and the weighted lengths between sections.

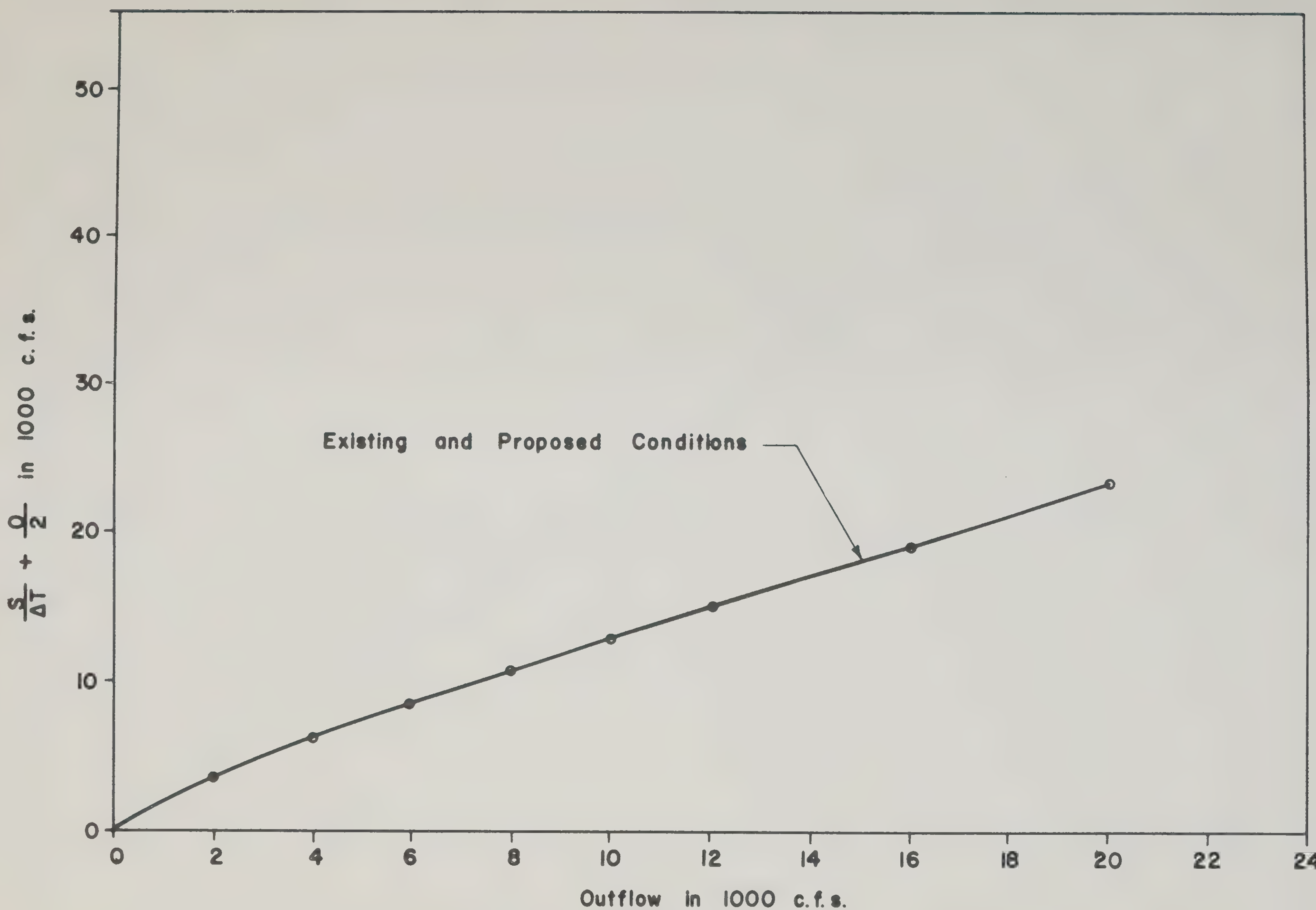
Knowing the relationship between discharge and storage for each reach, it was then possible to plot a curve showing the relationship between outflow and the storage indication factor $\frac{S}{\Delta t} + \frac{O}{2}$, which became a storage indication working curve for each reach. The storage indication working curves for each reach for both the proposed and existing conditions are shown on Figures 32 through 36.

The storage indication working curves shown on these figures made flood routing by use of the last equation quite simple as all known terms are on the left side of the equation, and all unknown terms on the right side of the equation.

Local Inflow:

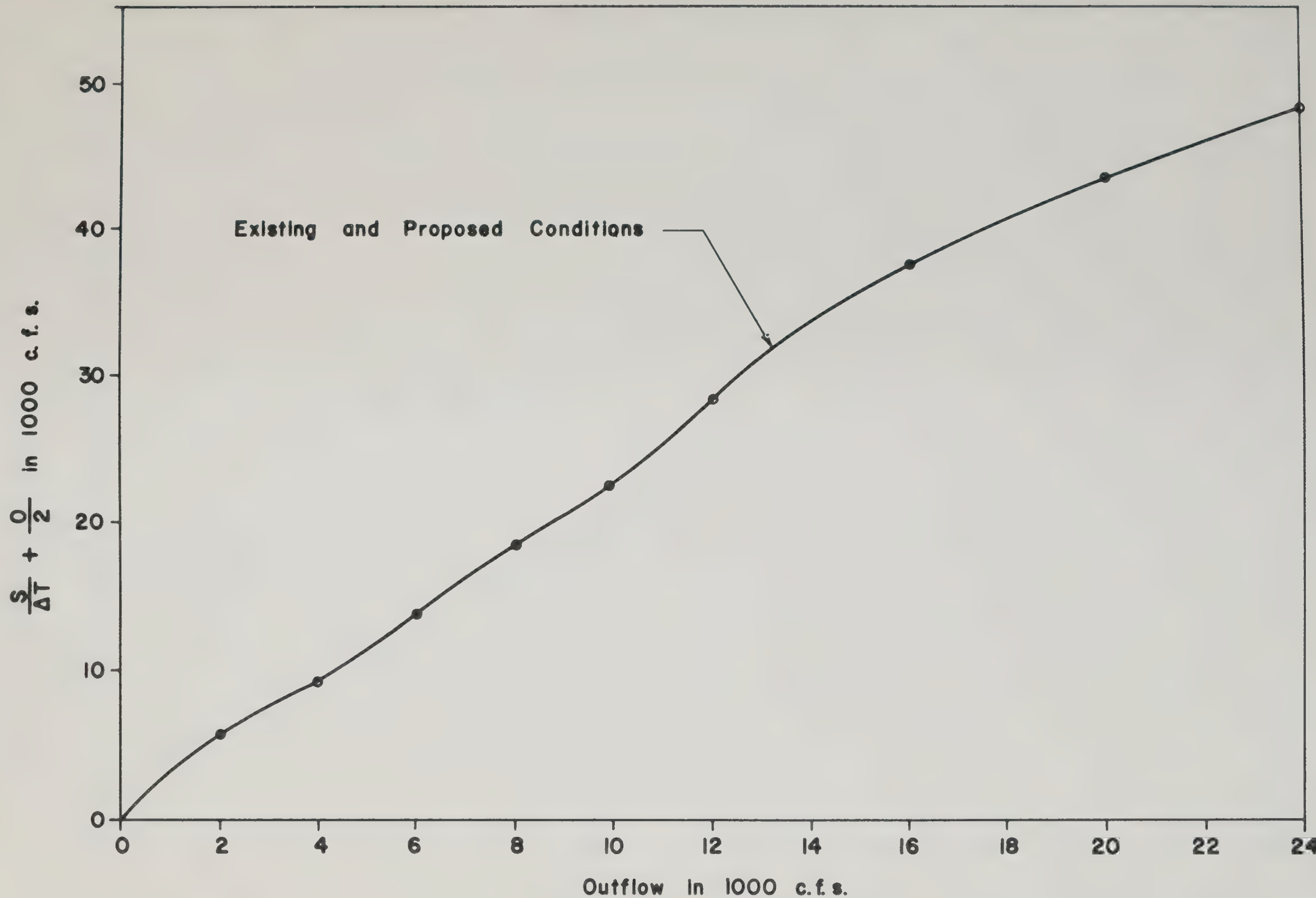
As mentioned herein, a design flood hydrograph was derived for each sub-drainage basin below the dam sites as shown on Figures 29 to 31. These design flood hydrographs were added as local inflow to the reach before flood routing, or to the outflow from the reach after flood routing, depending upon whether or not the major tributary, draining the local sub-drainage basin subtended by the particular reach, entered the main stem at the head or foot of the reach. That is to say, the entire tributary area of the particular sub-drainage basin was assummed to be concentrated at and enter either the head or foot of the particular reach, as previously mentioned.

The results of the flood routing performed in this report are shown on Figures 7 through 11.



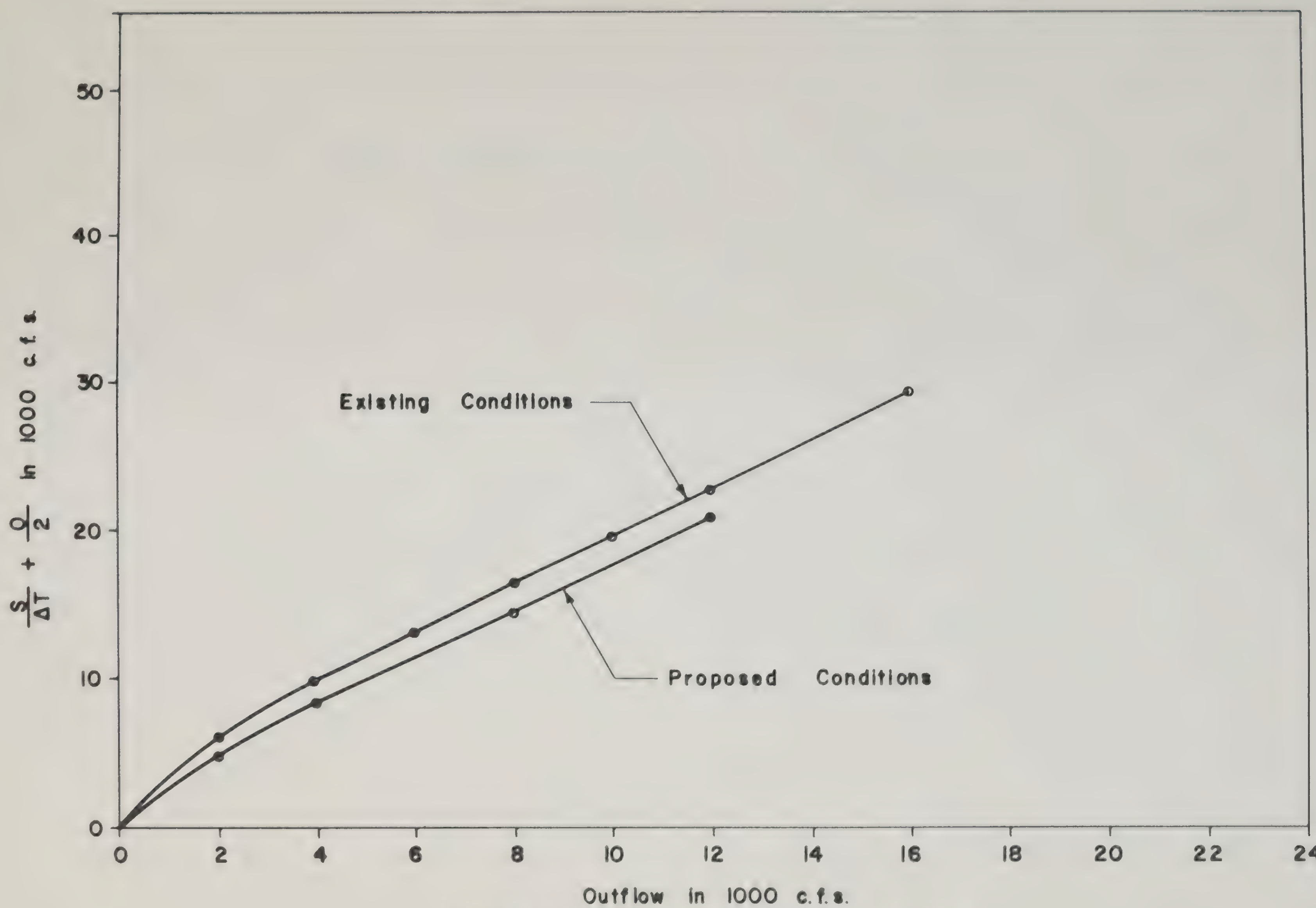
STORAGE - INDICATION CURVES
SOUTH BRANCH THAMES RIVER
MILE 6.4 TO MILE 10.8
(INGERSOLL TO BEACHVILLE)

FIGURE 32



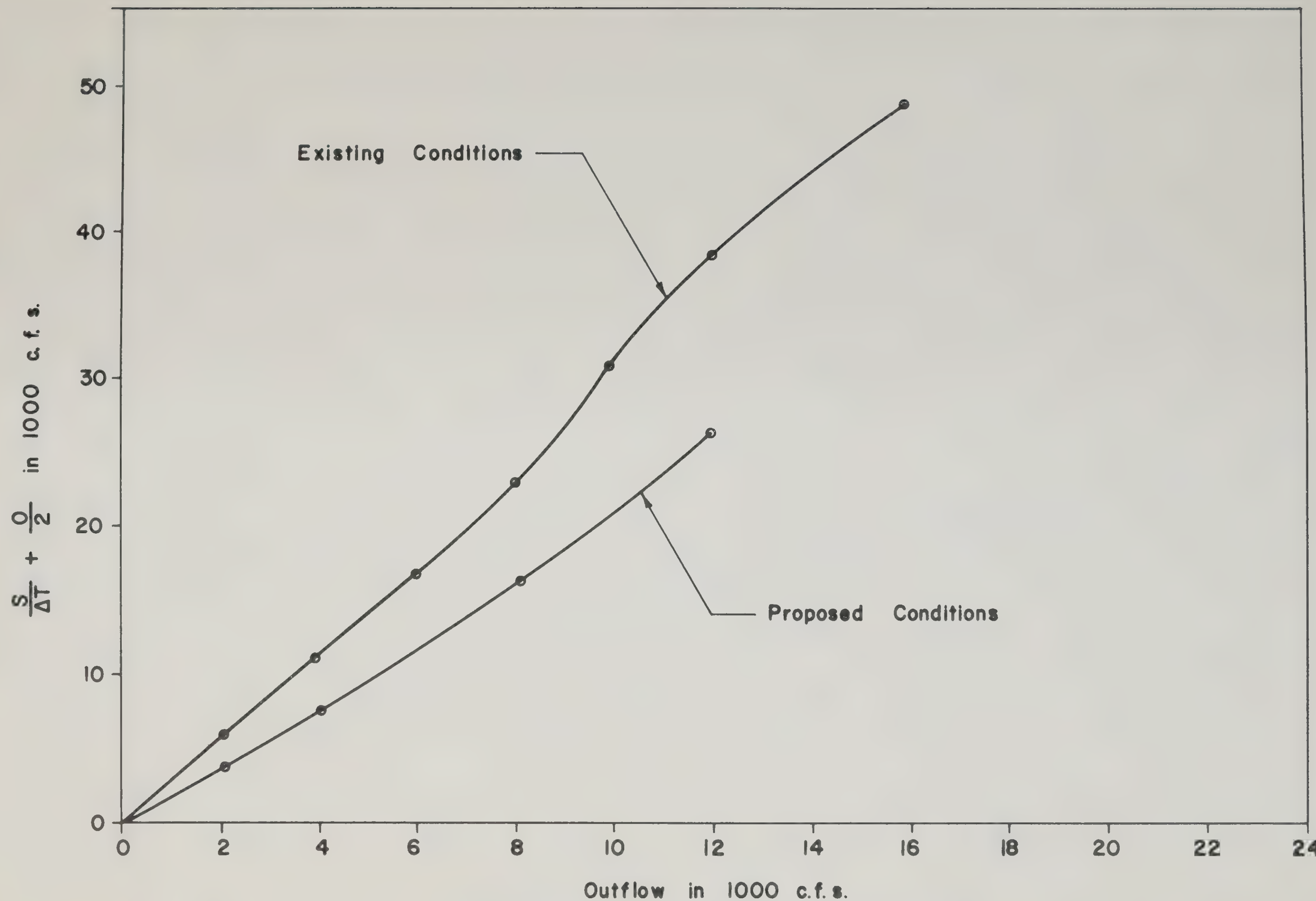
STORAGE – INDICATION CURVES
SOUTH BRANCH THAMES RIVER
MILE 10.8 TO MILE 13.0
(BEACHVILLE TO BEACHVILLE NORTH)

FIGURE 33



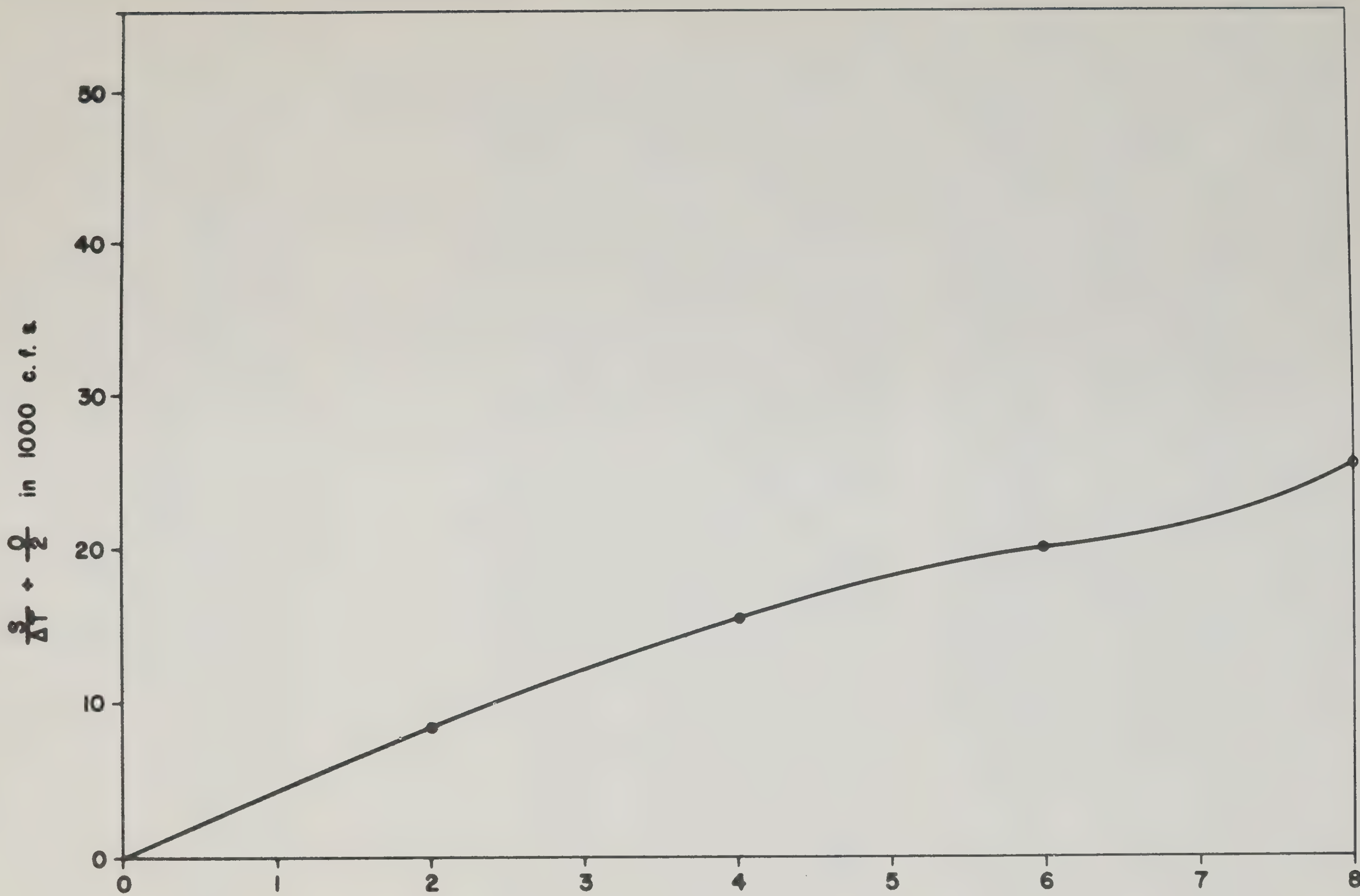
STORAGE – INDICATION CURVES
SOUTH BRANCH THAMES RIVER
MILE 13.0 TO MILE 15.4
(BEACHVILLE NORTH TO GOVERNOR'S ROAD BRIDGE)

FIGURE 34



STORAGE – INDICATION CURVES
SOUTH BRANCH THAMES RIVER
MILE 15.4 TO MILE 17.3
(GOVERNORS ROAD BRIDGE TO DAM SITE)

FIGURE 35



Outflow in 1000 c.f.s.

STORAGE — INDICATION CURVES
CEDAR CREEK
MILE 0.0 TO MILE 2.8
(SOUTH BRANCH THAMES RIVER TO DAM SITE)

FIGURE 36

RESERVOIR REGULATION

General:

Since the maximum available storage at each of the three reservoir sites under consideration is actually limited by the particular physical or economic characteristics of the reservoir areas, the functional operation of the reservoir under each alternate in order to achieve maximum flood control benefits with this available storage.

For both Alternate 1 and Alternate 2, a reservoir regulation method referred to by the U.S. Army Corps of Engineers [1] as "Method B - Regulation Based on Control of the Project Design Flood" was selected in order to show the effect of the reservoirs in reading flood peaks.

This method, being the most commonly used reservoir regulation method is based on the concept of controlling a particular project design flood. When actually used in the operation of reservoirs during a flood this method consists of releasing a previously established amount of water (or none), or of varying the rate of release, depending upon the reservoir contents, just as though the hypothetical project design storm was occuring. In this method of reservoir regulation previously established schedules are followed at all times on the assumption that the best over-all regulation of floods will result. Since, in most cases, the project design storm represents an unusual event, a fixed schedule for its regulation affords considerable assurance of satisfactory regulation of a large flood. This method of regulation is generally used where there is some definite downstream channel capacity available, such that little damage, if any, will occur at moderate flows.

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(1) "Reservoir Regulation" U.S. Army Corps of Engineers, Manual for Civil Works Construction, Part CXXXVI, August 1951, Washington, D.C.

This is the situation in the South Branch of the Thames River, where the construction of the proposed channel improvements near Woodstock will eliminate most of the minor damage caused by small floods (peak discharges of 5,000 to 10,000 cfs.). The application of this regulation procedure to the drainage basin of the South Branch of the Thames River is discussed herein.

The scope of this report is limited to the investigation of the effect of the dams, as proposed under both Alternates upon the peak of the selected project design flood at the quarry area between Ingersoll and Beachville, and upon the Town of Ingersoll, now protected to some degree by local channel improvement previously discussed in this report. The quarry area and the Town of Ingersoll, were considered to be the principal damage reach, or protected area.

Generally, when there is a substantial local inflow between the proposed dam (or dams) and the principal damage reach, the reservoir must be operated so as to produce a minimum peak at the principal damage reach or protected area, rather than a minimum peak at the dam. In such situations, the local inflow crests sooner than the inflow generated upstream and therefore, the operation usually requires little release, if any, early in the flood, with relatively higher releases toward the end of the flood. These high volume releases are timed to arrive at the damage reach after the peak of the local inflow has passed. This operating procedure was assumed for the two alternates presented. Under both alternates, the reservoirs were assumed to be empty, thus making all the storage volume in the proposed reservoir available to control the Project Design Flood.

In the operation of the dams under each alternate, it was assumed that towards the end of the flood, the gates would be slowly closed so that a full pool would be in existence in the reservoirs, when the project design flood ends.

Alternate 1 Reservoir Regulation:

The 13,400 acre-feet of reservoir storage available at this site, which is equivalent to 2.7 inches of runoff from the drainage basin above the dam, is of sufficient size to contain a large part of the design flood of 3.9 inches. Under Alternate 1, the most obvious method for operation of this single dam, following the general procedure and method outlined in the previous paragraphs, is to close the gates until the peaks of the local inflow hydrographs from downstream sub-drainage basins have passed. (e.g. Cedar Creek Tributary which effect peak flows down to Ingersoll).

After considerable study, the decision was made to keep the gates closed until 20 hours after runoff or rainfall-excess starts. At this time the reservoir level would be up to Elevation 945.4 and the reservoir would contain 8,700 acre-feet as shown on Figure 37. The gates would then be opened so as to allow 4,000 cfs to pass over the spillway. At the 32nd. hour after the start of runoff, the reservoir would be full, (Maximum Pool Elevation 950.0)

The operator must then start to close the gates so that the rate of discharge over the spillway generally follows the recession curve of the inflow hydrograph. At the 70th. hour after the start of the runoff, the gates will be completely closed. Under these operating conditions, the reservoir will be at full pool at the conclusion of the design project storm.

Using the above assummed sequence of reservoir regulation and operation, the peak discharge resulting from the project design flood, at various downstream points will have been considerably reduced, as shown on Figures 7 through 11.

Alternate 2 Reservoir Regulation.

The 7,200 acre-feet of storage available at the proposed Cedar Creek Dam Site, which is equivalent to 4.1 inches of runoff from the drainage basins above the dam, is of sufficient size to contain all of the design flood of 3.9 inches shown on Figure 31. With this large amount of storage available, the most obvious operational procedure for this dam is to close the gates of the dam at the start of storm and to keep them closed until the flood is over. This action completely cuts off most of the Cedar Creek tributary, which is the largest sub-drainage basin, and thus the major source of local flow below the proposed Woodstock Dam on the South Branch of the Thames River, thus making operation and regulation of the proposed Woodstock Dam much easier.

The 5,000 acre-feet of storage available at the Woodstock Dam Site on the South Branch of the Thames River, near Woodstock, as proposed under Alternate 2, is equivalent to only 1.2 inches of runoff from the drainage basin as compared with 3.9 inches of runoff from the design flood. However, since most of the sub-drainage basins below this dam site are quite small, with the large Cedar Creek Tributary being almost completely eliminated by the proposed Cedar Creek Dam, it was quite obvious that a fairly large

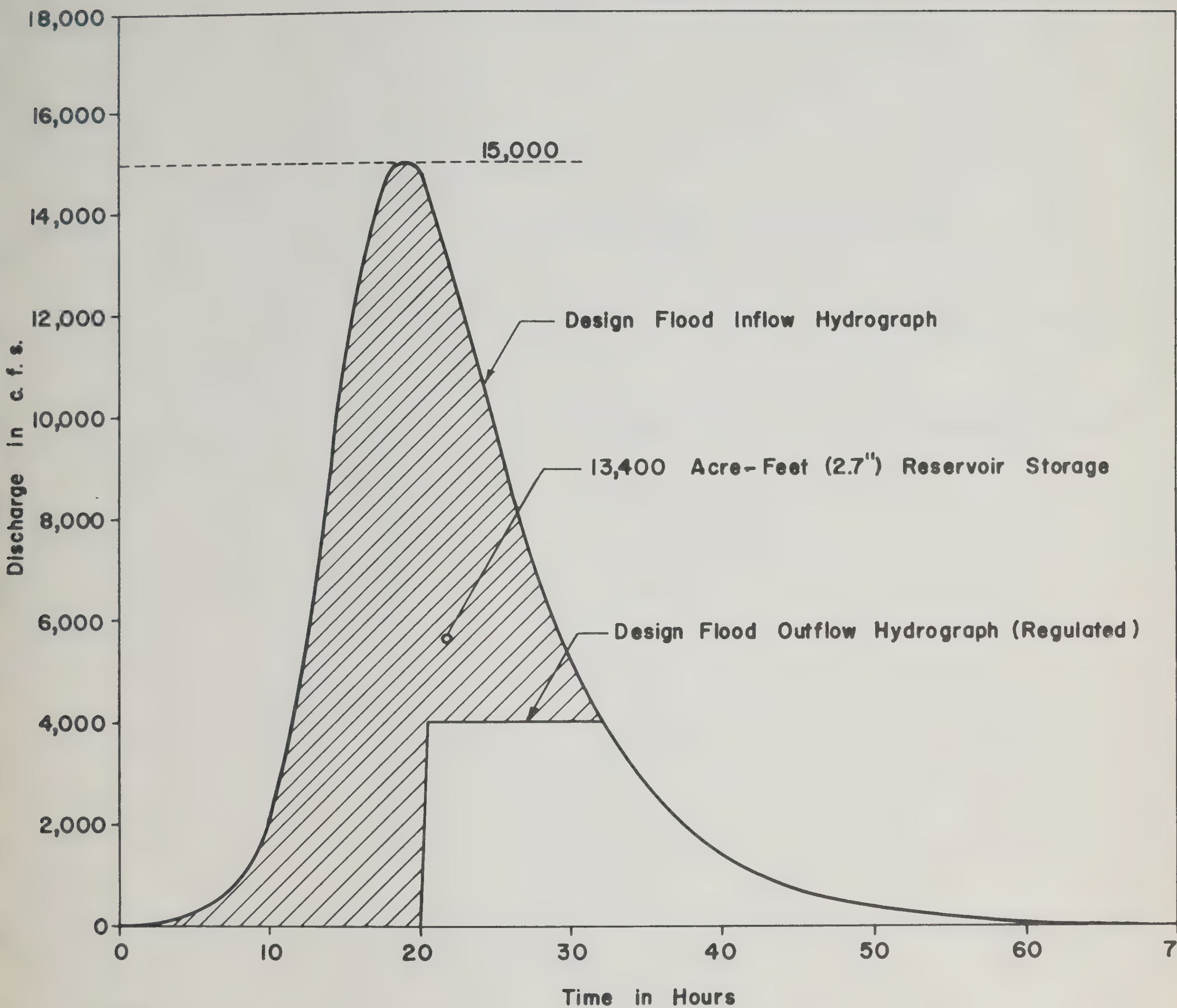
initial flow from this proposed dam was allowable and could be coincident with the peak flow of the local inflow hydrographs, while remaining contained within the banks of the existing channel improvement, at the damage reach between Beachville and Ingersoll. From a study of the situation, the descision was made to operate the gates so as to discharge 4,500 cfs until 19 hours after runoff begins, at which time the water level in the reservoir would be at Elevation 937.8 and the reservoir would contain 3,820 acre-feet as shown on Figure 38.

At this time, after the peaks of the remaining local inflow hydrographs have passed, the gates would then be opened up further and would be operated to discharge 10,500 cfs over the spillway. A gate opening time of 0.5 of an hour was assumed for this report. At the 24th. hour after the start of runoff, the reservoir would be full (Maximum Pool Elevation 940.0)

In order to keep the reservoir full, so that water for a low flow maintenance pool is available after the flood is over, the operator would then begin to close the gates so that the rate of discharge over the spillway will generally follow the recession curve of the inflow hydrograph, as was previously mentioned under Alternate 1. At the end of the 70th hour the gates are assumed to be completely closed.

The actual outflow hydrograph line between the origin and the 19th hour, as shown on Figure 38 has been idealized to a certain degree. In order to discharge flow over the spillway, even with all of the gates wide open, a certain amount of head must be built up over the spillway, which in turn, automatically causes water to be stored in the reservoir. However, probably no significant error was introduced by making this straight line assumption.

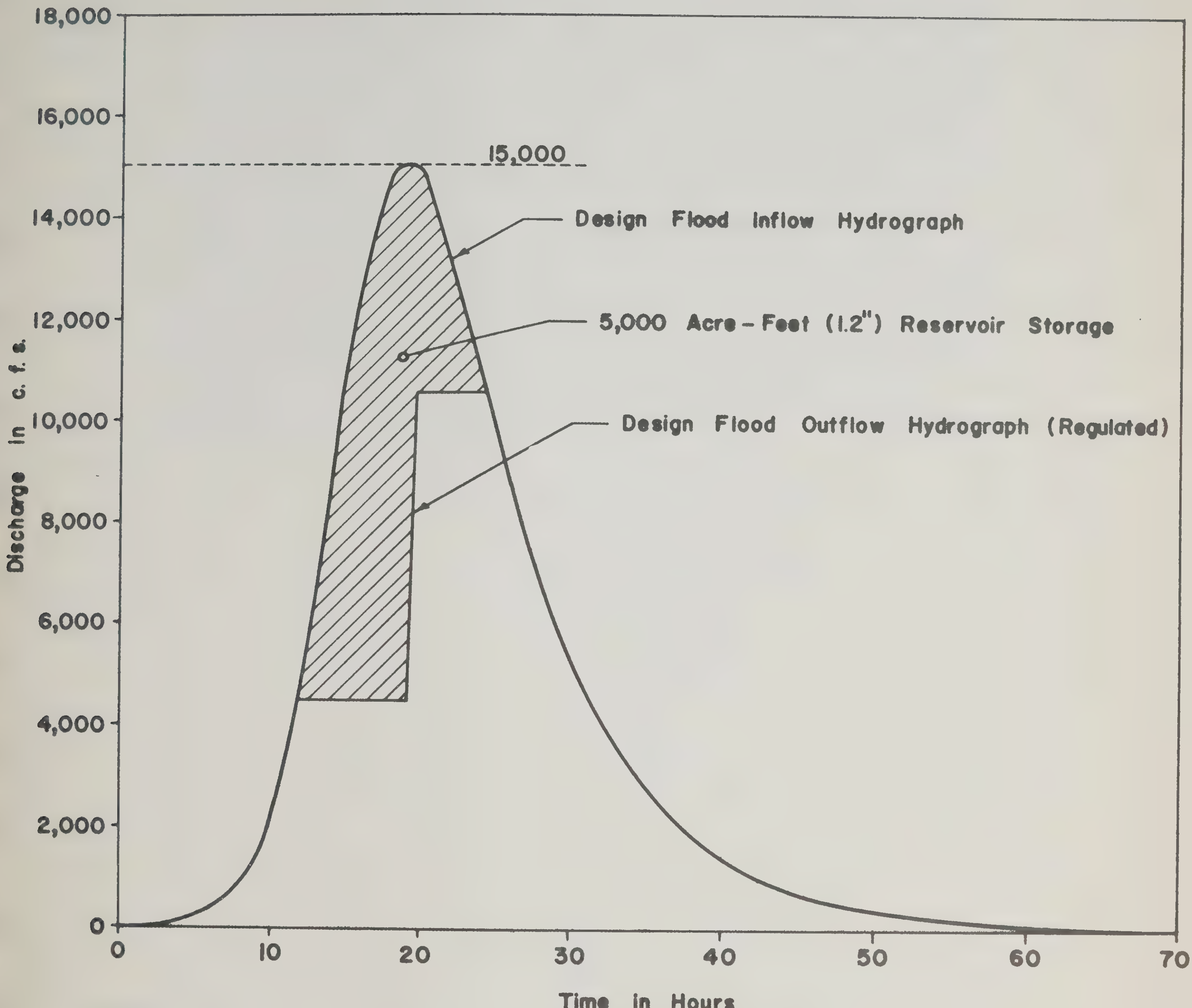
Using the above assumed sequence of reservoir regulation and operation, the peak discharges of thehydrographs, resulting from the project design storm, have been considerably reduced at downstream points by the two reservoirs proposed under Alternate 2. This reduction in peak discharge is shown on Figures 7 through 11. From these figures, a slight advantage of this Alternate, in reducing peak discharge at these downstream points, over the single dam proposed under Alternate 1, is noticeable.



NOTE:
0.5 Hour Gate Operation Assumed

DESIGN FLOOD INFLOW & OUTFLOW HYDROGRAPHS
WOODSTOCK DAM
SOUTH BRANCH THAMES RIVER
ALTERNATE I — HIGH DAM
(PARTIAL GATE OPENINGS)

FIGURE 37



NOTE:
0.5 Hour Gate Operation Assumed

DESIGN FLOOD INFLOW & OUTFLOW HYDROGRAPHS
WOODSTOCK DAM
SOUTH BRANCH THAMES RIVER
ALTERNATE 2 — LOW DAMS
(PARTIAL GATE OPENINGS)

FIGURE 38

The final descision as to the actual method of operation of the reservoir finally constructed, so as to control both small and large floods, should be made only after sufficient rainfall and river gauging stations have been in operation for a period of time and after much study has been completed, and a reservoir regulation section is in operation. For instance if the dams as proposed under Alternate 2 were always operated in the manner described in the previous paragraphs, there is a possibility that during smaller floods than the Project Design Flood that the sudden opening of the gates of the proposed Woodstock Dam to discharge 10,500 cfs, when the water level is up to elevation 937.8, may result in a flood wave larger than the actual peak of that flood, under the existing condition of no dams in place. The high dam on the South Branch of the Thames River, as proposed under Alternate 1, could probably be safely operated in the manner previously described, most of the time.

Maximum Release Rate:

As was discussed herein, the assumption was made under both Alternates that the reservoir would be empty at the start of the project design storm (all the reservoir storage would be available to control floods). This situation is in direct conflict with the other proposed uses of the dam, e.g.(providing a storage area for a low flow maintenance pool and providing a source of water for irrigation). These other uses require that the reservoir be kept as full as possible at all times in order to provide water for these purposes. Therefore, at the spring of the year, or upon the approach of a storm which is expected to cause major flooding conditions in the drainage basin, all water contained by the dam at that time should be dumped in order to provide this necessary flood control storage. However, this stored water in the reservoir cannot be dumped quickly, even though spillway capacity may be available, due to the probable occurence of a damaging flood wave.

Generally, the science of meterology has advanced sufficiently so that a 48 hour advance warning of a major flood producing rainfall can usually be made. Therefore, 48 hours was assumed to be the maximum allowable time to dump the reservoirs for the purposes of this report.

In order to dump a possible full pool of 13,400 acre-feet from the reservoir, in a two day period, as proposed under Alternate 1, it would be necessary to operate the gates so that a discharge over the spillway would be at the rate of about 3,500 cfs. This is not excessive and could be fully contained in and do very little damage to the proposed channel improvements to the South Branch of the Thames River, near Woodstock, as described in another section of this report, or to the existing channel improvement between Beachville and Ingersoll. In the region between Beachville and Woodstock, there would probably be some minor flooding (overbank flow) as a result of this proposed release rate, since the dry weather flow channel in this area is quite small. Since this land at present is used only for pasture, probably no flood damage would occur.

Under Alternate 2, the release rate from the Cedar Creek Dam should be kept as small as possible, due to the fact that the channel capacity of Cedar Creek in the Woodstock area is quite low. A release rate of 1,750 cfs would be required to empty a possible full reservoir in the maximum allowable two day period, and would probably cause some very minor flooding and inconvenience, but no flood damage, in the vicinity of Southside Park.

The Woodstock Dam on the South Branch of the Thames River could be fully emptied of 5,000 acre-feet at a release rate of only 1,250 cfs in the maximum allowable two day period. When combined with the 1,750 cfs from Cedar Creek Dam, a total rate of discharge of 3,000 cfs could pass down the river and again without causing other than very minor flooding in the area between Beachville and Woodstock.

COMPUTATION OF MAXIMUM PROBABLE FLOOD (SPILLWAY HYDROGRAPH)

In the computation of the maximum probable flood for the proportionment of the spillways of the dams proposed in the drainage basin of the South Branch of the Thames River, a hydro-meteorological approach was used. This approach consists of first computing the maximum probable precipitation and then computing the maximum probable flood by application of the maximum probable runoff (maximum probable precipitation-infiltration) to a unit hydrograph of the particular drainage basin.

Estimates of the maximum probable precipitation that could reasonably be expected over the drainage basins above the proposed dam sites on the South Branch of the Thames River and on Cedar Creek, were taken from generalized precipitation curves for Southern Ontario, prepared by Bruce [1]. These curves enable the computation of the maximum probable precipitation, resulting from both thunderstorm and tropical storms, for drainage basins varying in size from 10 to 5,000 square miles.

From these curves, it was immediately apparent that the thunderstorm situation caused the maximum probable precipitation in the drainage basins above both dam sites and that the 12 hour time period was critical. The maximum probable rainfall as obtained from these curves for both drainage basins, as arbitrarily increased by 5% due to the lower latitude, as recommended by Bruce, are listed in Table 25 below.

Table 25

Maximum Probable Rainfall

Drainage Basin	Area Sq. Miles	6 hr. Rainfall inches	12 hr. Rainfall inches
South Branch of the Thames River	93.5	15.3	15.6
Cedar Creek	33.2	16.4	16.7

(1) "Preliminary Estimates of Probable Maximum Precipitation over Southern Ontario" by J. P. Bruce, The Engineering Journal, July 1957.

The maximum 6 hour period in each case was broken down in accordance with the accumulated 2 hour percentages, 64%, 84%, and 100%, as recommended by the United States Bureau of Reclamation(1), since the derived synthetic unit hydrographs for both of the above drainage basins were of two hour duration. The 2 hour rainfall increments were thus rearranged, in arbitrary order, as shown in the hyetographs on Figures 39 and 40.

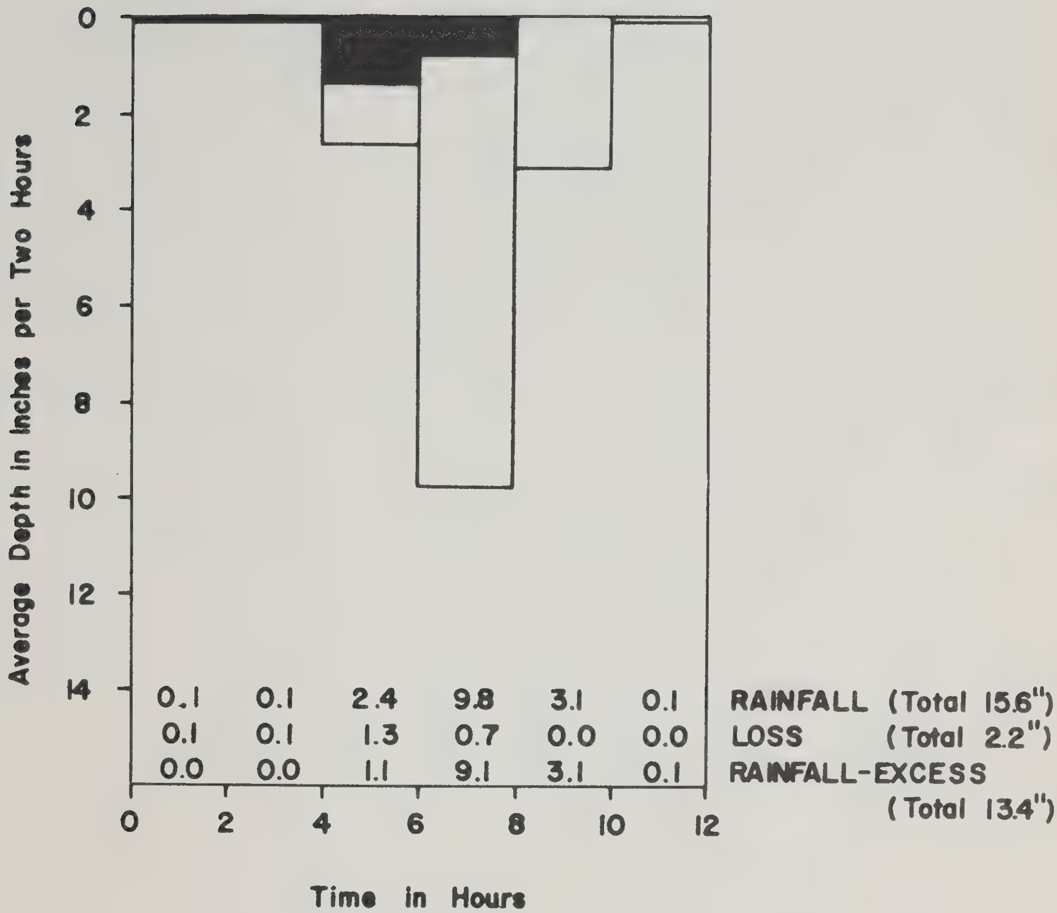
In the computation of the maximum probable flood runoff (rainfall-excess), resulting from this maximum possible precipitation, Antecedent Moisture Condition 2 was assumed to be the soil moisture condition prior to the occurrence of the storm for both alternates, because of the extremely rare likelihood of a storm of this size occurr-ingwith the thoroughly wet soils condition assumed with Antecedent Moisture Condition 3. Runoff curve No. 82 was used to compute this Maximum Probable Flood Runoff for both alternate sites, since the drainage basins above the dam sites, in each case, were in Hydrologic Soil Group C, as previously discussed and as indicated in Tables 18 and 19.

Hyetographs of the average rainfall, rainfall-excess or volume and losses for the Maximum Probable Precipitation over each of the drainage basins is shown on Figures 39 and 40.

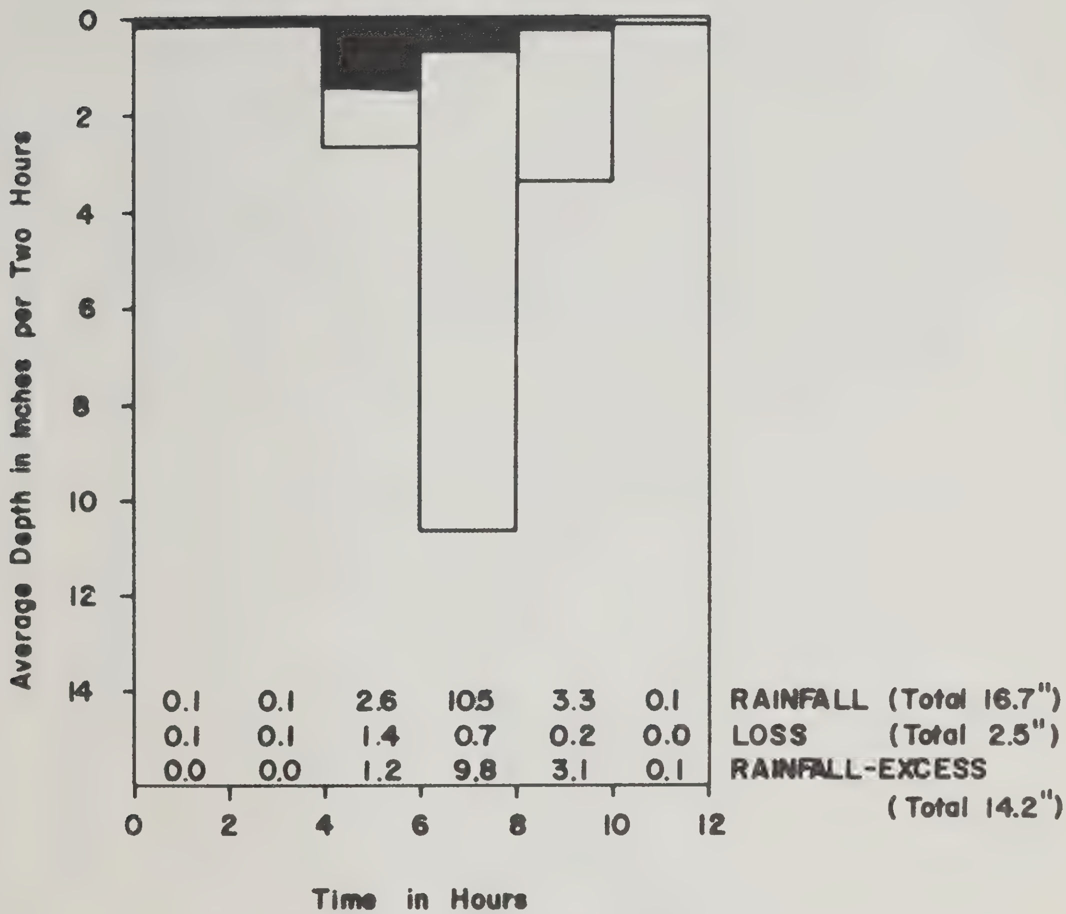
For each basin, the Maximum Probable Flood was computed by the application of the maximum probable flood runoff to the respective unit hydrograph. The Maximum Probable Flood (Spillway Hydrographs) for the drainage basin above each dam site is shown on Figures 41 and 42.

---

(1) "Design of Small Dams". United States Department of the Interior, Bureau of Reclamation, United States Government Printing Office, Washington, D.C. 1960, pge 50.



HYETOGRAPH OF MAXIMUM PROBABLE STORM
SOUTH BRANCH THAMES RIVER AT DAM SITE
(Soil Type C, Drainage Area = 93.5 sq. miles)

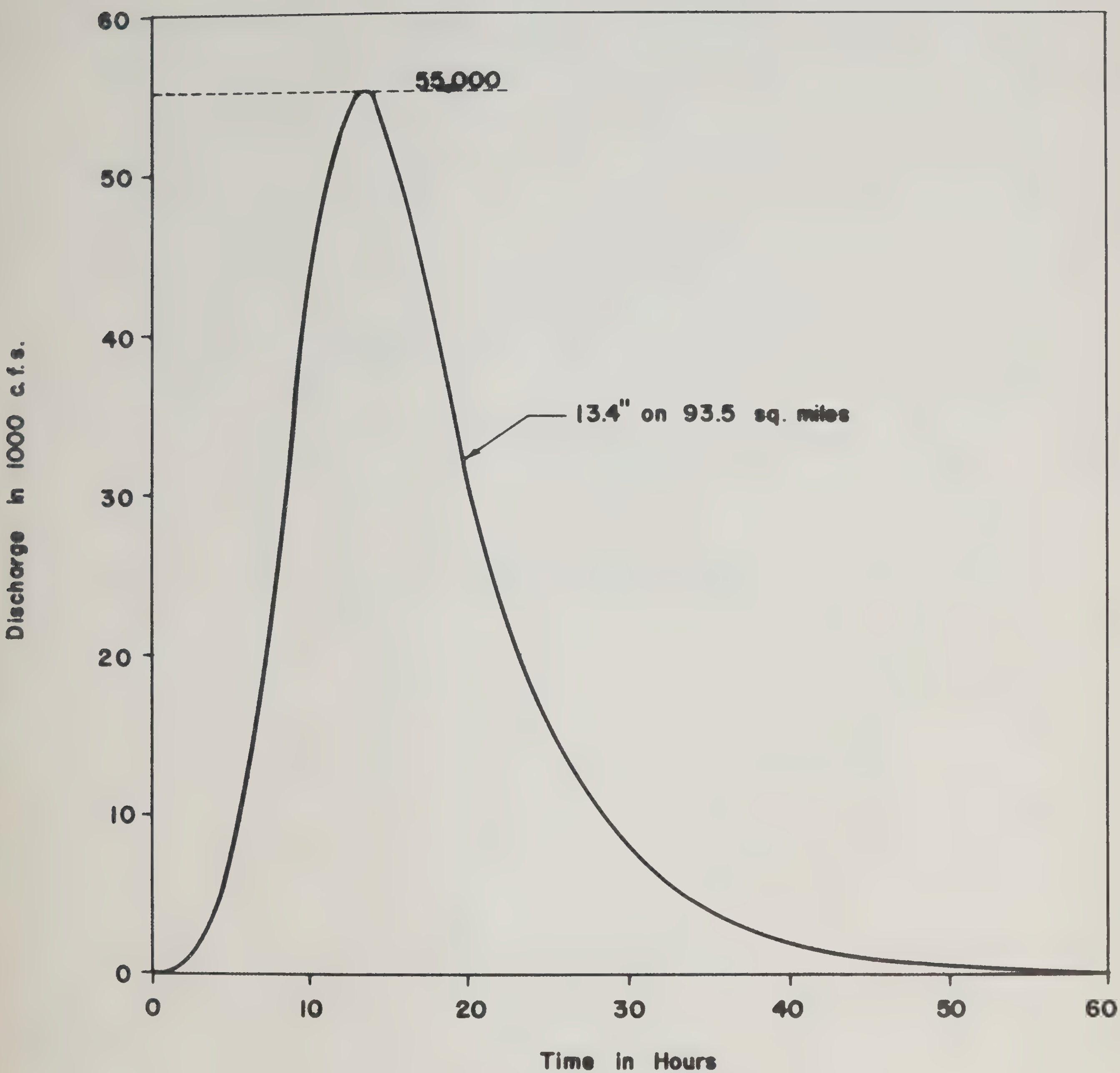


HYETOGRAPH OF MAXIMUM PROBABLE STORM

CEDAR CREEK AT DAM SITE

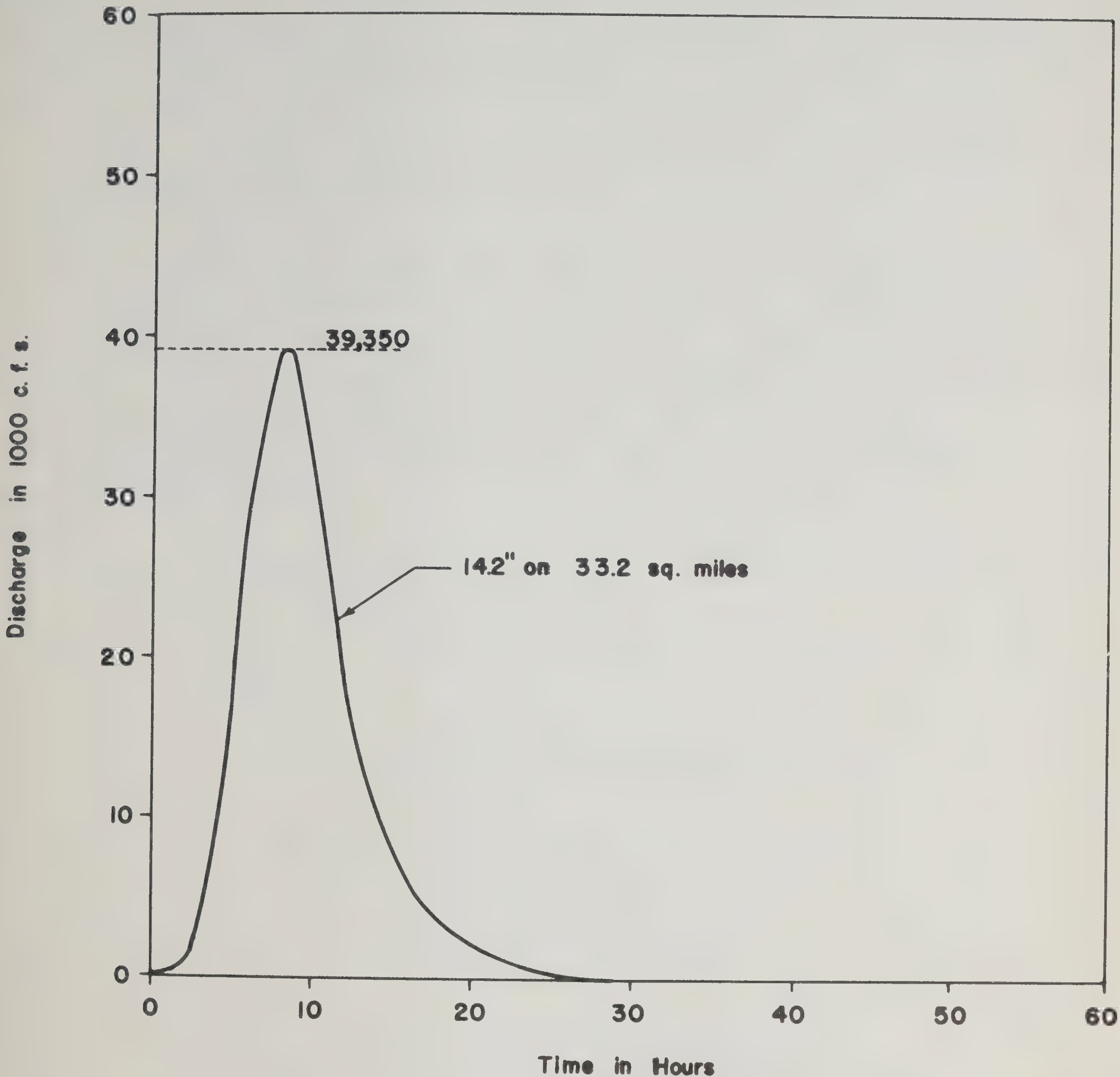
(Soil Type C , Drainage Area = 33.2 sq. miles)

FIGURE 40



MAXIMUM PROBABLE FLOOD
(SPILLWAY HYDROGRAPH)
SOUTH BRANCH THAMES RIVER
(AT WOODSTOCK DAM SITE)

FIGURE 41



MAXIMUM PROBABLE FLOOD
(SPILLWAY HYDROGRAPH)
CEDAR CREEK
(AT CEDAR CREEK DAM SITE)

FIGURE 42

Water surface profiles (backwater curves) were computed on the South Branch of the Thames River and on Cedar Creek for various selected discharge for the purpose of measuring the storage volumes beneath the water surface at each discharge, to determine the areal extent of flooding at each discharge and to establish tailwater rating curves at the base of each dam. Water surface profiles were computed for both the existing and proposed(with channel improvements) conditions on both rivers.

The actual method used in the computation of the water surface profiles on the Thames River and Cedar Creek was similar to Method A of the United States Bureau of Reclamation [(1)]. This method like all other methods used to compute water surface profiles involved the use of Bernoulli's equation in which an energy balance is made between two points on the river. The elevation at the downstream point is known or assumed and the hydraulic elements of the cross-section at each point such as area, wetted perimeter and hydraulic radius, have been computed.

In this method, the friction slope at each section is computed by use of the following equation:

$$S_f = \frac{(Q)^2}{K_d}$$

where

S_f = friction slope in feet per foot
Q = discharge in cubic feet per second
K_d = conveyance.

The conveyance (K_d) is defined by the equation

$$K_d = \frac{1.486}{n} AR^{2/3}$$

where

n = Manning's roughness coefficient
A = cross-sectional area in square feet
R = hydraulic radius in feet

The average friction loss (in feet) was computed by taking the average of the friction slope at each section and then multiplying it the length between sections.

The following other assumptions were made in the computation of the water surface profile.

(1) "Guide for Computing Water Surface Profiles", Sedimentation Section Report, United States Department of the Interior, Bureau of Reclamation, Denver, Colorado, Nov. 1957, pge 24.

1. A single Mannings roughness coefficient was assigned to an entire cross section. No effort was made to separate the channel and overbank flow due to the fact that the dry weather flow channel was very small and almost all of the capacity is in the overbank region.

2. The starting elevations of the water surface profiles in the South Bank of the Thames River were the computed normal depths in the existing channel improvement between Beachville and Ingersoll.

3. Eddy losses in converging reaches were assumed to be 10% of the difference in velocity heads of the section at the head and foot of the reach when the differences in velocity heads was one foot or more. Eddy losses in diverging reaches were assumed to be 20% of the difference in velocity heads at the sections when the difference in velocity heads was 0.5 feet or more.

4. Head losses due to bends in the river were reflected by an increase in the Manning's roughness coefficient of 0.005.

5. Head losses through bridges were generally computed by using a method described in the publication "Hydraulics of Bridge Waterways" (1), except when the bottom of the bridge was submerged by high water in which case an orifice condition was assumed. In certain instances, where high discharges in the river completely overtopped the bridge, an orifice and weir condition was assumed.

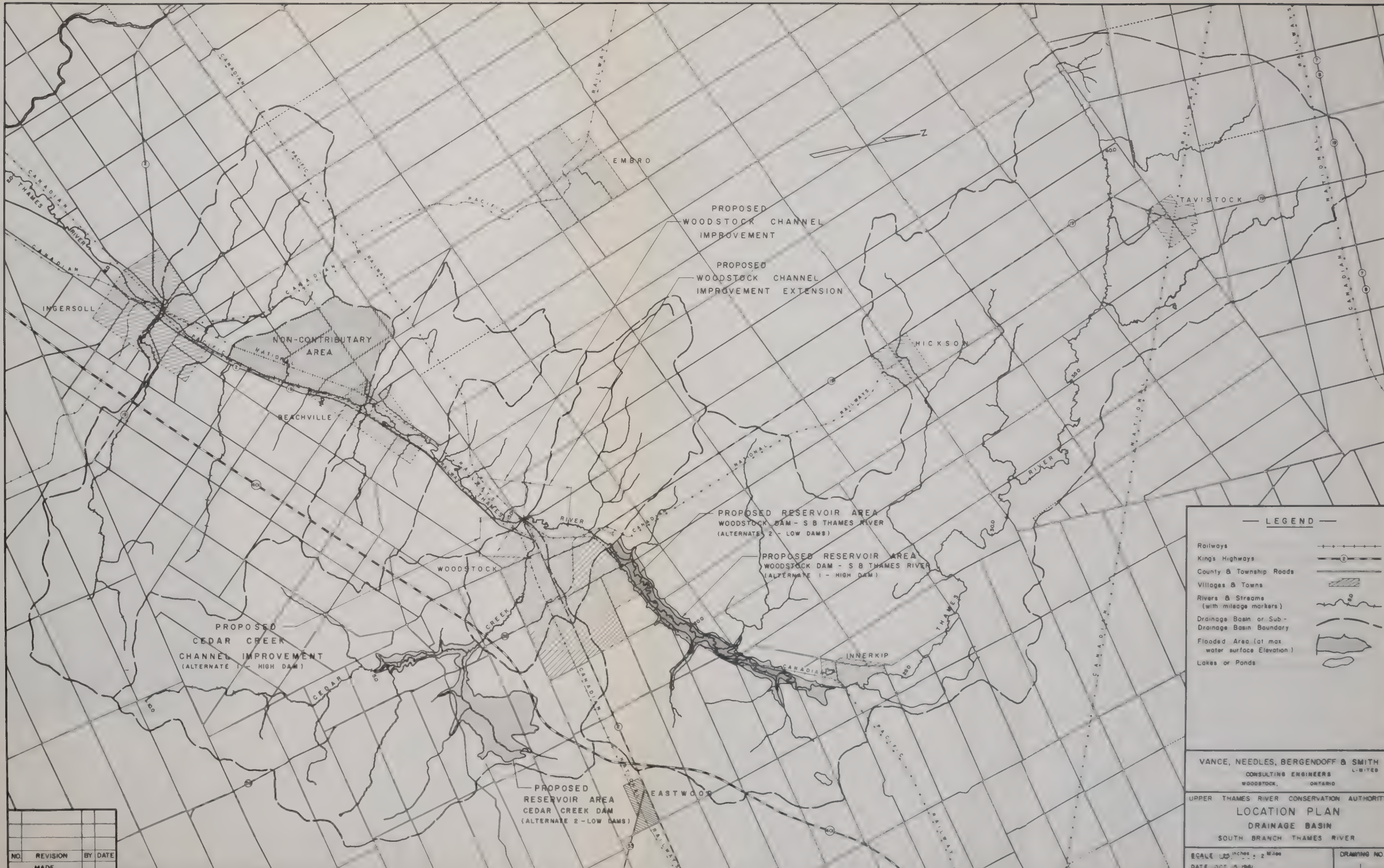
The computed water surface profiles on the South Branch of the Thames River for both the proposed and existing conditions are shown on Drawings 32 through 35. The computed water surface profiles of Cedar Creek for both the proposed and existing conditions are shown on Drawings 36 through 38.

---

(1) "Hydraulics of Bridge Waterways" United States Department of Commerce, Bureau of Public Roads, U.S. Government Printing Office, Washington, D.C. 1960.

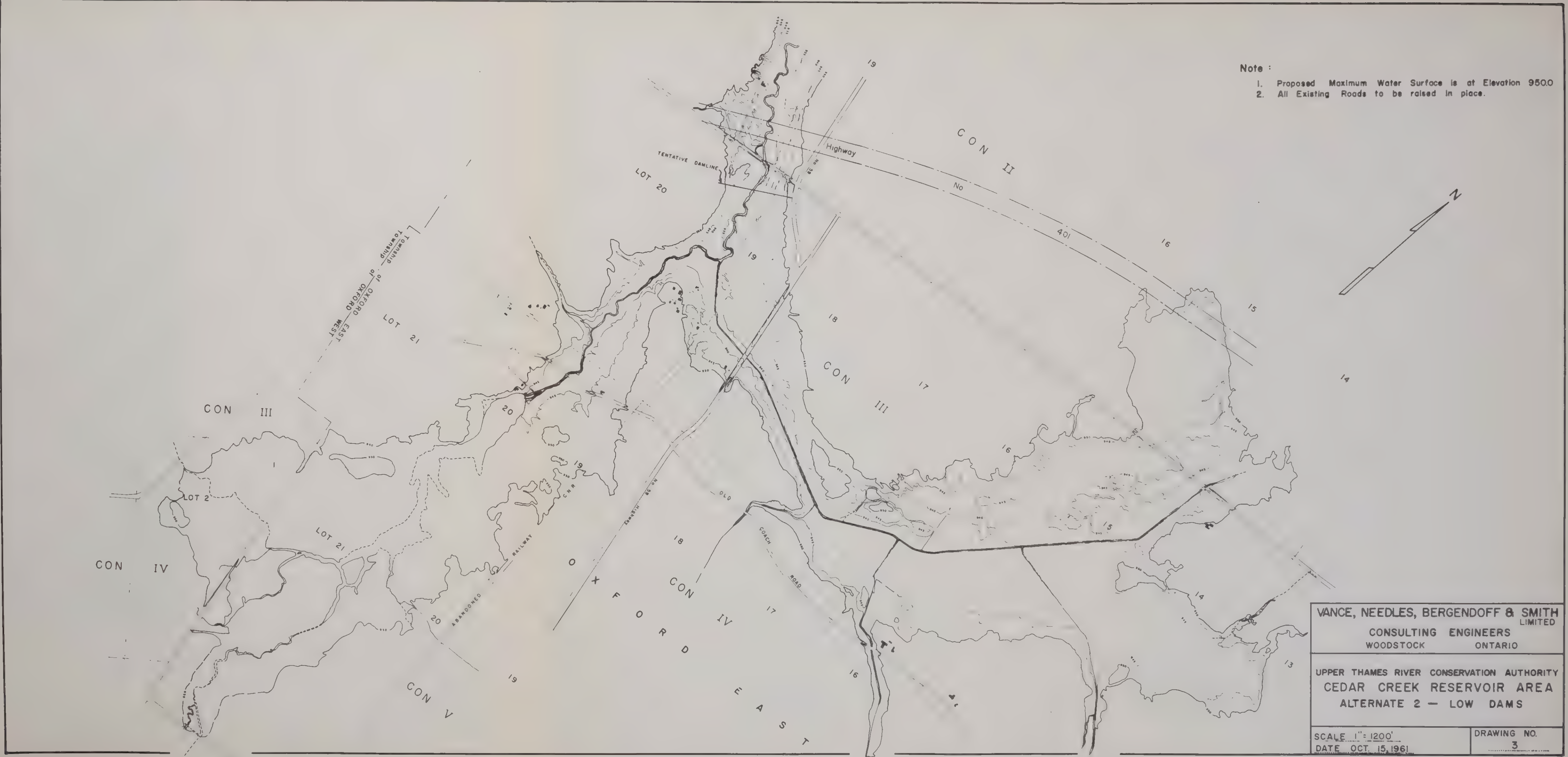
LIST OF FIGURES

LIST OF DRAWINGS

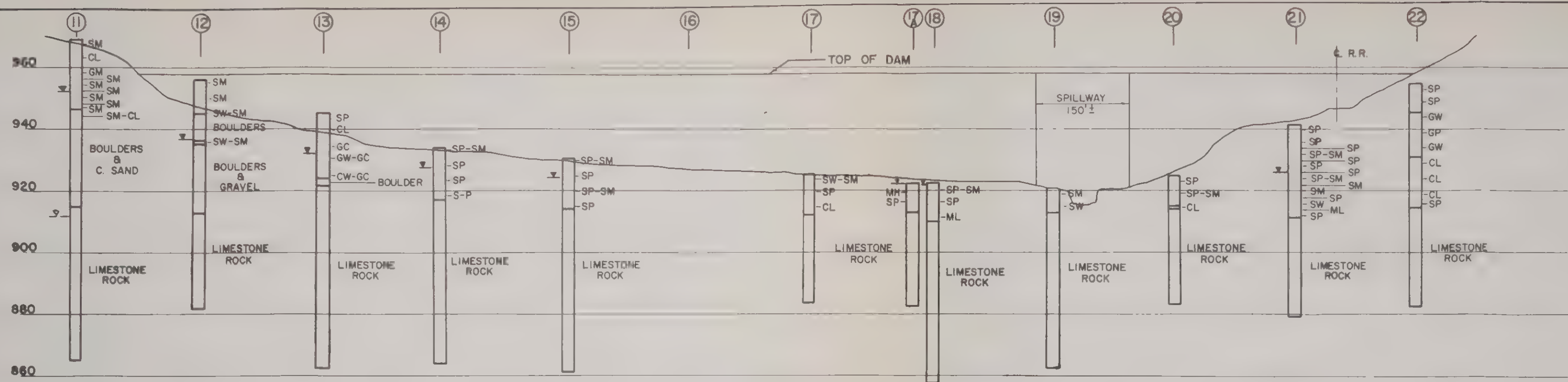


EMBRO
PROPOSED
WOODSTOCK CHANNEL
IMPROVEMENT
PROPOSED
WOODSTOCK CHANNEL
IMPROVEMENT EXTENSION
TAVISTOCK
INGERSOLL
NON-CONTRIBUTARY
AREA
HICKSON
BEACHVILLE
PROPOSED RESERVOIR AREA
WOODSTOCK DAM - S B THAMES RIVER
(ALTERNATE 2 - LOW DAMS)
PROPOSED RESERVOIR AREA
WOODSTOCK DAM - S B THAMES RIVER
(ALTERNATE 1 - HIGH DAM)
WOODSTOCK
PROPOSED
CEDAR CREEK
CHANNEL IMPROVEMENT
(ALTERNATE 1 - HIGH DAM)
INNERKIP
PROPOSED
RESERVOIR AREA
CEDAR CREEK DAM
(ALTERNATE 2 - LOW DAMS)
EASTWOOD
LEGEND
Railways
King's Highways
County & Township Roads
Villages & Towns
Rivers & Streams
(with mileage markers)
Drainage Basin or Sub-
Drainage Basin Boundary
Flooded Area (at max
water surface Elevation)
Lakes or Ponds
VANCE, NEEDLES, BERGENDOFF & SMITH
LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO
UPPER THAMES RIVER CONSERVATION AUTHORITY
LOCATION PLAN
DRAINAGE BASIN
SOUTH BRANCH THAMES RIVER
SCALE inches = 2 Miles
DATE OCT
DRAWING NO.
1
NO.
REVISION
MADE
BY
DATE

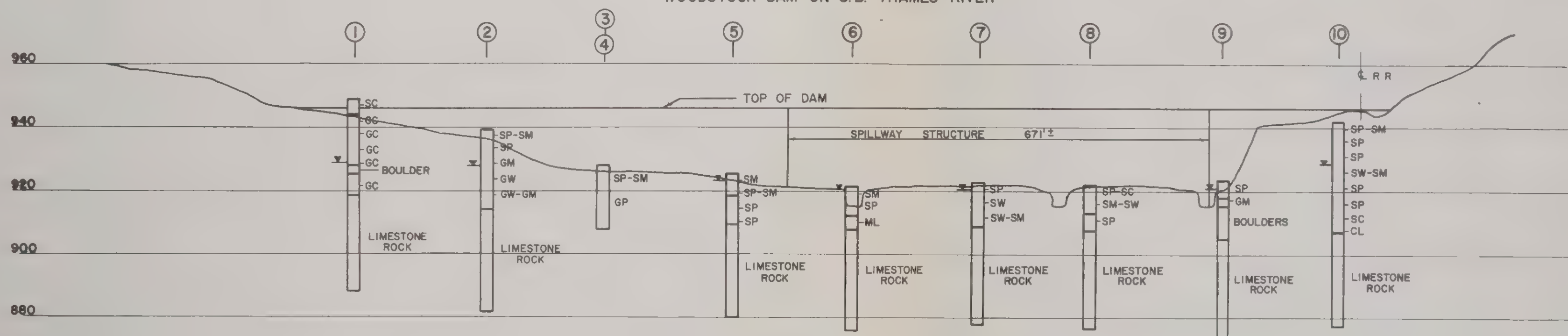

℄ Proposed Low Dam
℄ Proposed High Dam
N
S. B Thames River
Innerkip Road
(County Road 4)
N
MATCH LINE
MATCH LINE
Existing Canadian Pacific Railway
Huron Street
King's Highway No. 19
Note ·
Proposed Maximum Water Surface Elevation of the two proposed resevoirs is as follows
Alternate 1 — Elevation 9500
Alternate 2 — Elevation 9400
VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK ONTARIO
UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK RESERVOIR AREA
ALTERNATE 1 — HIGH DAM
ALTERNATE 2 — LOW DAMS
SCALE 1" = 1000'
DATE OCT 15, 1961
DRAWING NO. 2



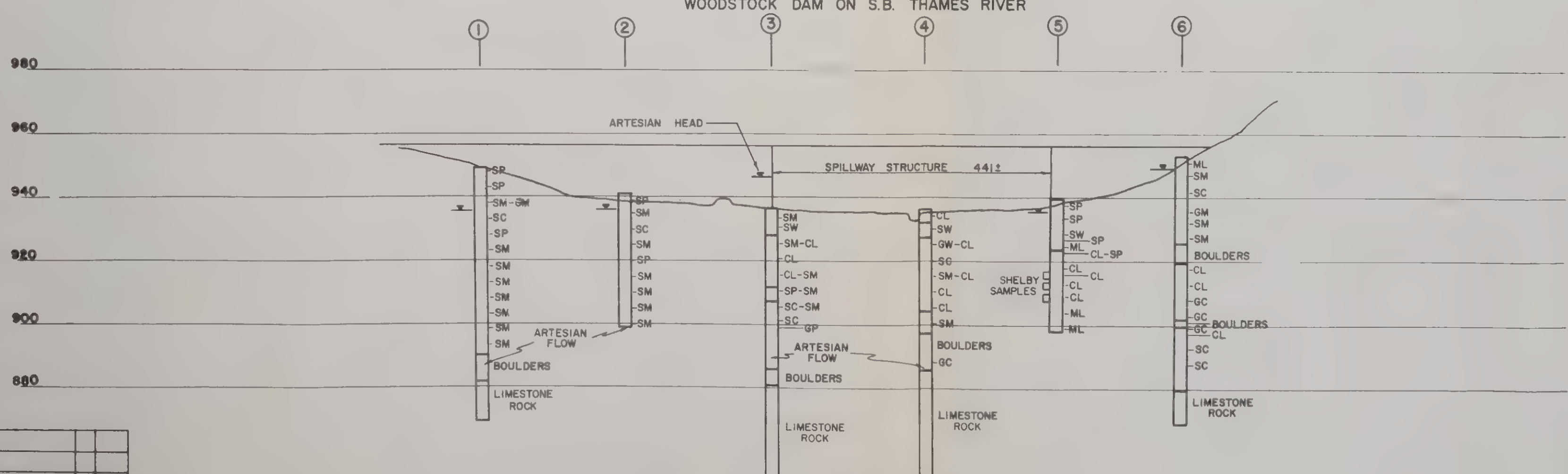
Note :
1. Proposed Maximum Water Surface is at Elevation 950.0
2. All Existing Roads to be raised in place.
CON II
Highway No 401
TENTATIVE DAMLINE
LOT 20
LOT 21
Township of OXFORD WEST
Township of OXFORD EAST
CON III
CON IV
CON V
LOT 2
LOT 21
ABANDONED RAILWAY
C.N.R.
OLD COACH ROAD
OXFORD EAST
N
VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK ONTARIO
UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK RESERVOIR AREA
ALTERNATE 2 — LOW DAMS
SCALE 1" = 1200'
DATE OCT. 15, 1961
DRAWING NO. 3



ALTERNATE 1 – HIGH DAM
WOODSTOCK DAM ON S.B. THAMES RIVER



ALTERNATE 2 – LOW DAMS
WOODSTOCK DAM ON S.B. THAMES RIVER



ALTERNATE 2 – LOW DAMS
CEDAR CREEK DAM ON CEDAR CREEK

GENERAL NOTES

1. All soils are classified in accordance with the United Soils Classification System
2. The symbol denotes observed height of the ground water table.
3. The symbol denotes a disturbed sample.
4. The symbol denotes an undisturbed sample.
5. All cased hole borings through overburden were $2\frac{7}{8}$ inches in diameter (BX Casing) except boring numbers 2 and 5 at the Cedar Creek Dam site which were $4\frac{1}{2}$ inches in diameter and numbers 3 and 4 at the Woodstock Dam site which were 8 inches in diameter.
6. All core borings in rock were $1\frac{1}{8}$ inches in diameter.

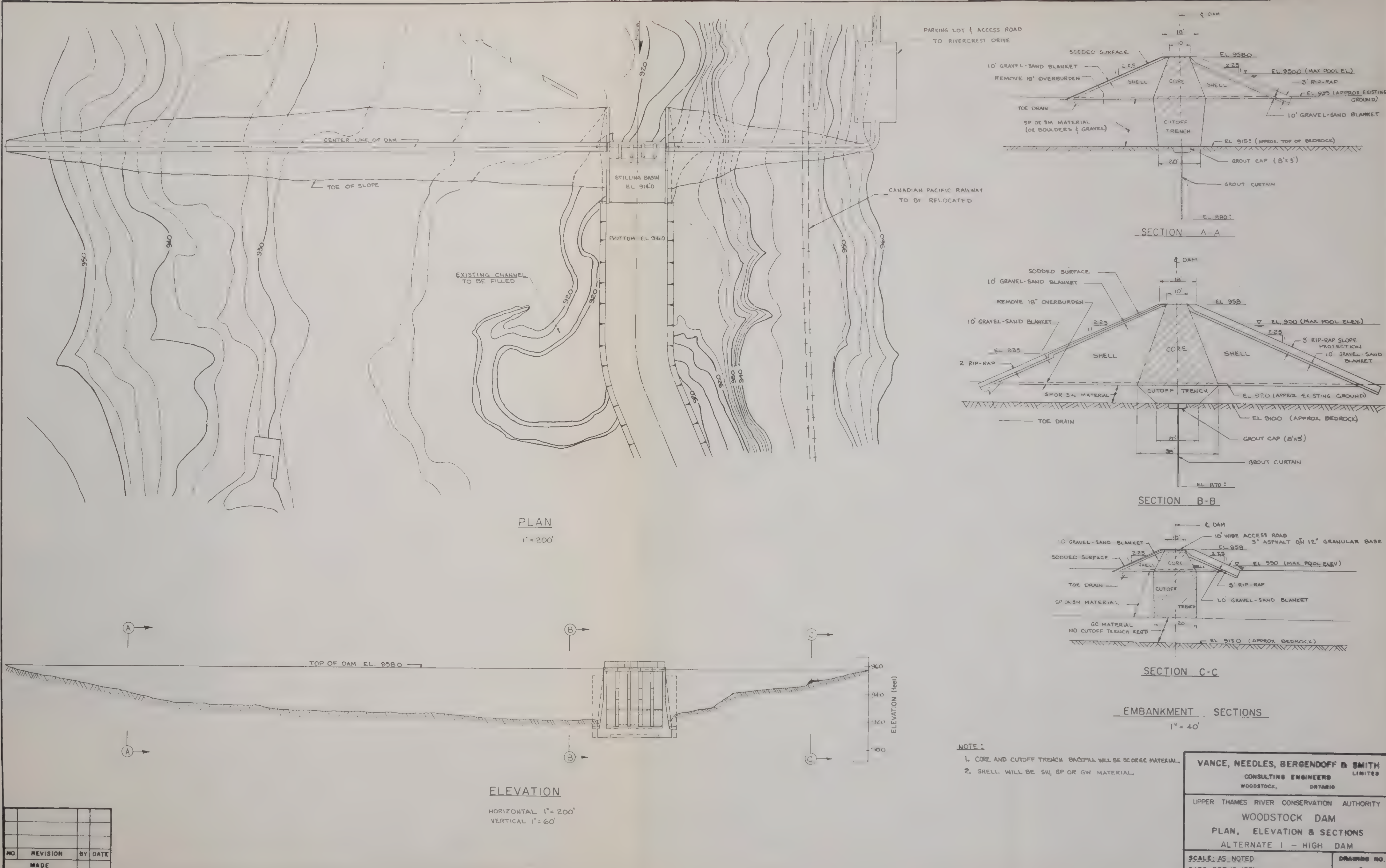
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

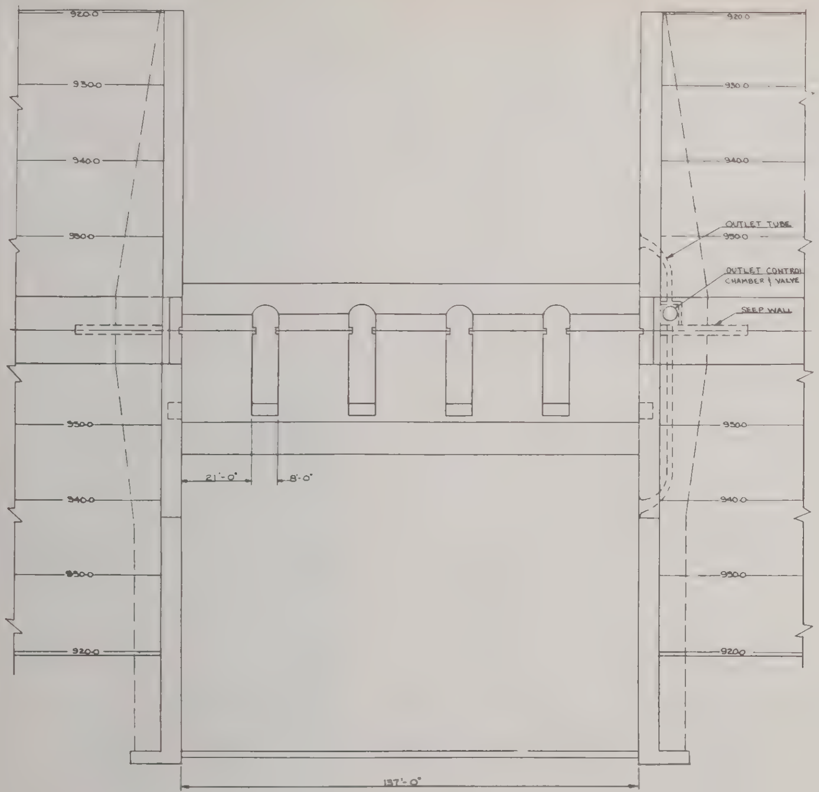
UPPER THAMES RIVER CONSERVATION AUTHORITY

SOIL & ROCK PROFILES
ALTERNATE 1 – HIGH DAM
ALTERNATE 2 – LOW DAMS

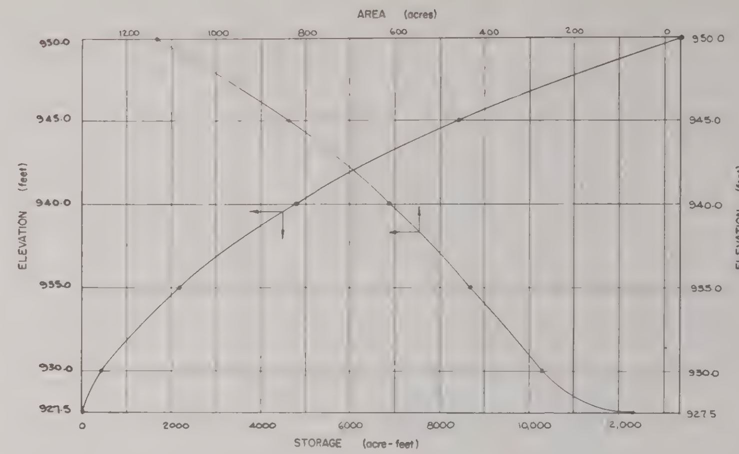
SCALE: Hor 1" = 200' Vert. 1" = 40'
DATE: OCT 15, 1961
DRAWING NO. 4



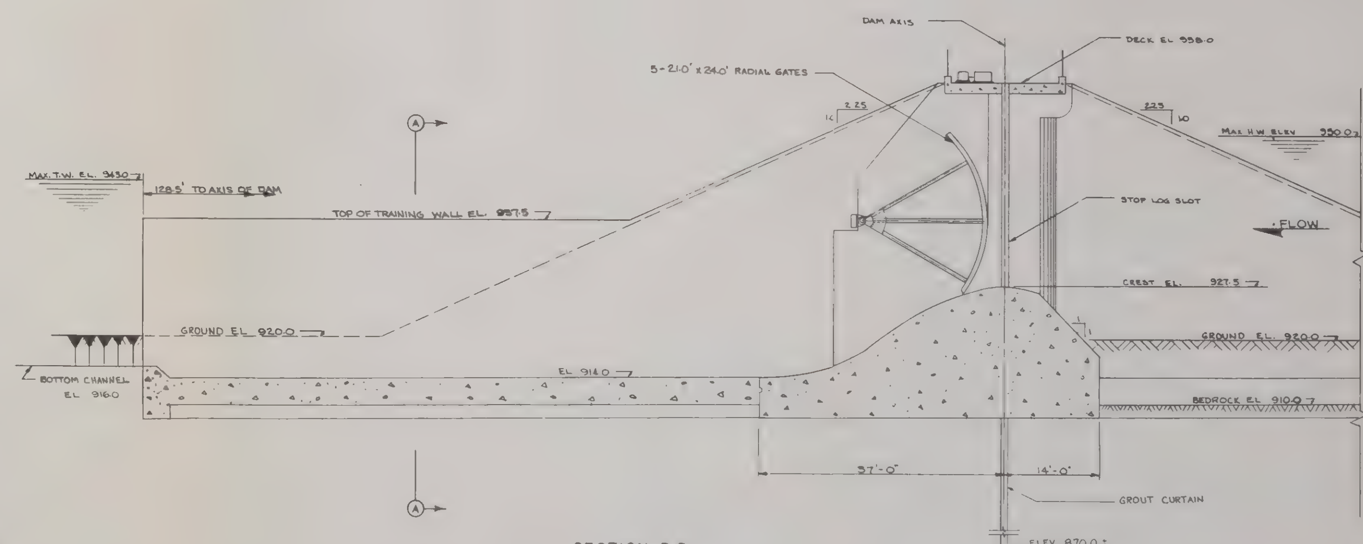
PARKING LOT & ACCESS ROAD TO RIVERCREST DRIVE
CENTER LINE OF DAM
TOE OF SLOPE
STILLING BASIN EL 914.0
BOTTOM EL 916.0
EXISTING CHANNEL TO BE FILLED
CANADIAN PACIFIC RAILWAY TO BE RELOCATED
950
940
930
920
960
PLAN
1" = 200'
TOP OF DAM EL. 958.0
ELEVATION (feet)
ELEVATION
HORIZONTAL 1" = 200'
VERTICAL 1" = 60'
SODDED SURFACE
10' GRAVEL-SAND BLANKET
REMOVE 18" OVERBURDEN
TOE DRAIN
SP OR SM MATERIAL (OR BOULDERS & GRAVEL)
SHELL
CORE
CUTOFF TRENCH
EL 958.0
EL 950.0 (MAX POOL EL)
3' RIP-RAP
EL 935 (APPROX EXISTING GROUND)
EL 915± (APPROX. TOP OF BEDROCK)
GROUT CAP (8'x3')
GROUT CURTAIN
EL 880±
SECTION A-A
EL 958
EL 950 (MAX POOL ELEV.)
3' RIP-RAP SLOPE PROTECTION
EL 935
2' RIP-RAP
SP OR SW MATERIAL
EL 920 (APPROX EXISTING GROUND)
EL 910.0 (APPROX BEDROCK)
GROUT CAP (8'x3')
EL 870±
SECTION B-B
10' WIDE ACCESS ROAD 3" ASPHALT ON 12" GRANULAR BASE
GC MATERIAL NO CUTOFF TRENCH REQ'D
EL 913.0 (APPROX BEDROCK)
SECTION C-C
EMBANKMENT SECTIONS
1" = 40'
NOTE:
1. CORE AND CUTOFF TRENCH BACKFILL WILL BE SC OR GC MATERIAL.
2. SHELL WILL BE SW, SP OR GW MATERIAL.
VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO
UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
PLAN, ELEVATION & SECTIONS
ALTERNATE I - HIGH DAM
SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 5
NO. REVISION MADE BY DATE



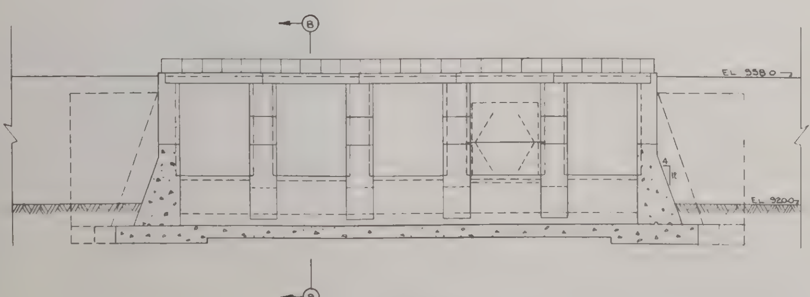
PLAN
(BRIDGE NOT SHOWN)
1" = 40'



AREA STORAGE CURVES



SECTION B-B
1" = 20'



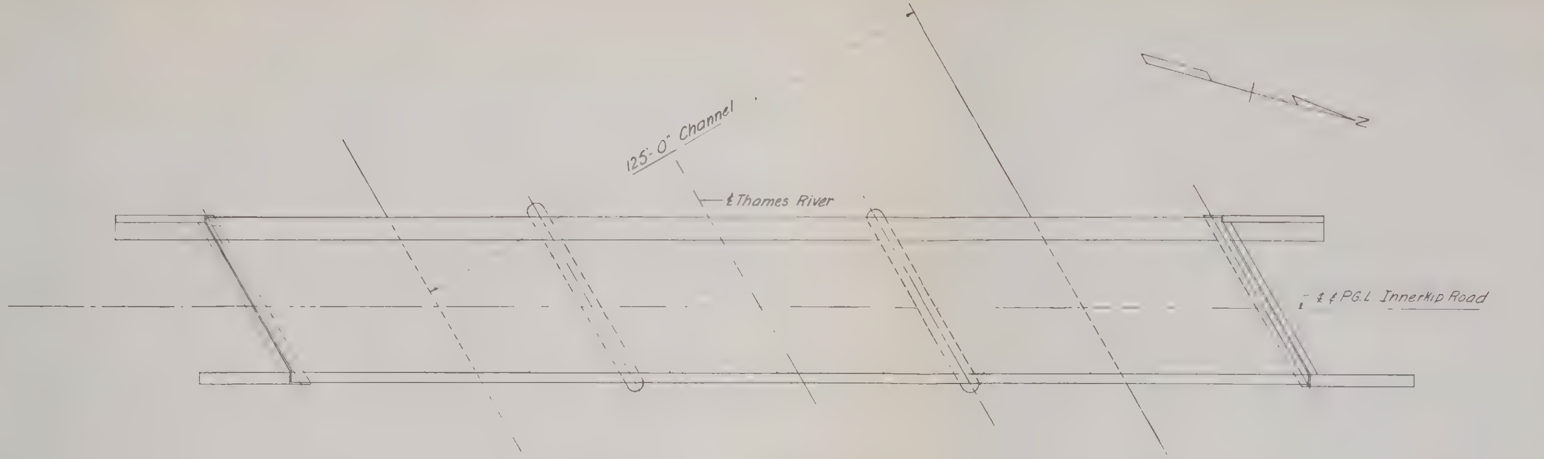
ELEVATION
SECTION A-A
(ONE GATE SHOWN)
1" = 40'

NO.	REVISION MADE	BY	DATE

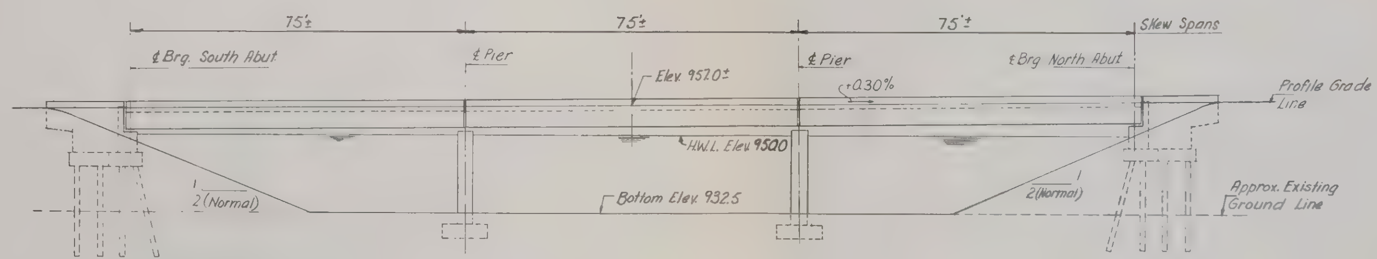
VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
DETAILS OF SPILLWAY SECTION
ALTERNATE I - HIGH DAM

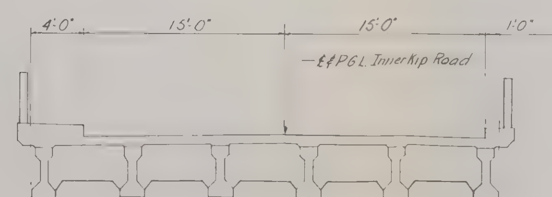
SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 6



PLAN
1"=30'



ELEVATION
1"=30'



TYPICAL CROSS SECTION
1"=10'-0"

NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
INNERKIP ROAD BRIDGE OVER THAMES RIVER
PLAN & ELEVATION
ALTERNATE 1 – HIGH DAM

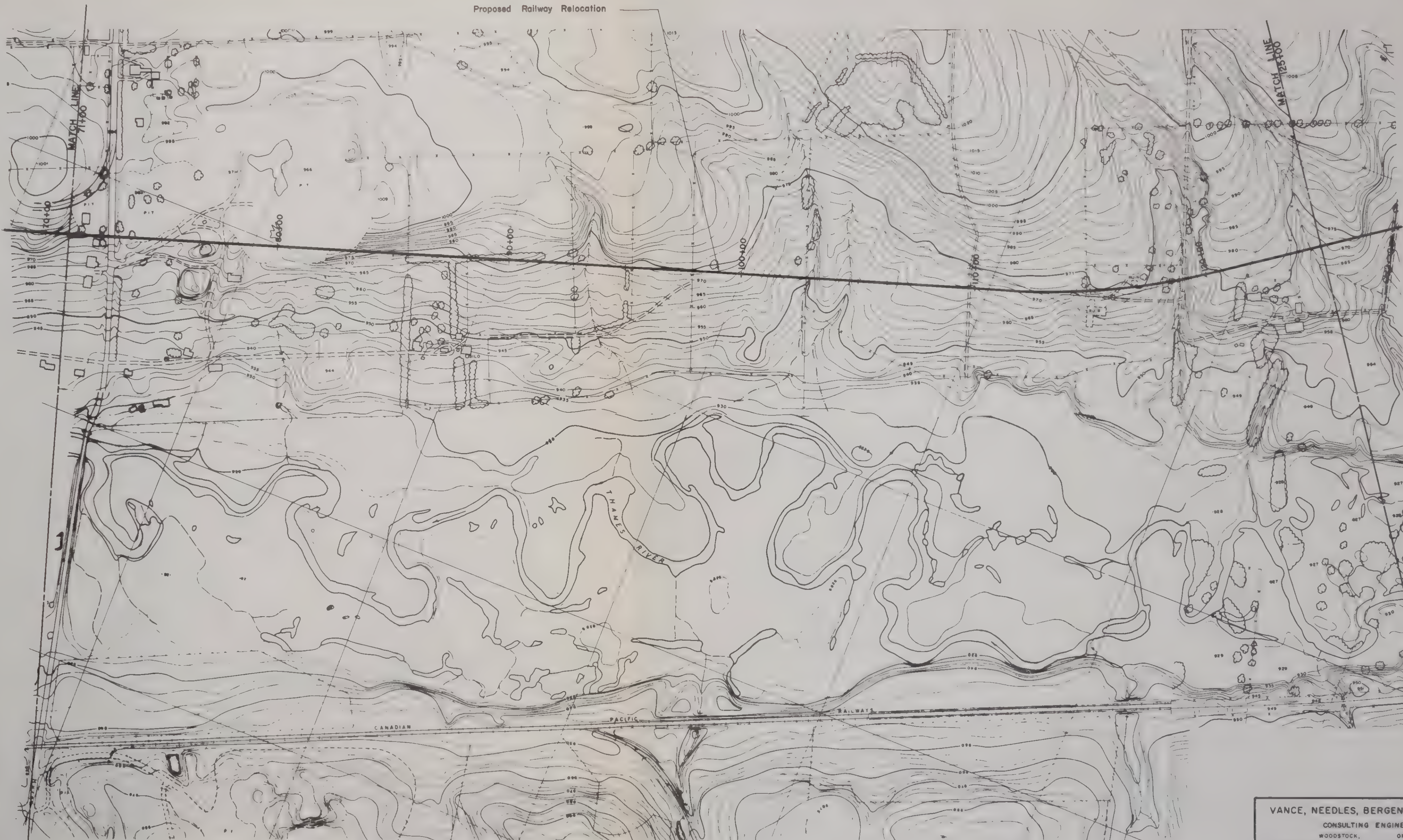
SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 7

NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CANADIAN PACIFIC RAILWAY RELOCATION
PLAN
ALTERNATE 1 - HIGH DAM

SCALE: 1" = 400'
DATE: OCT. 15, 1961
DRAWING NO. 8

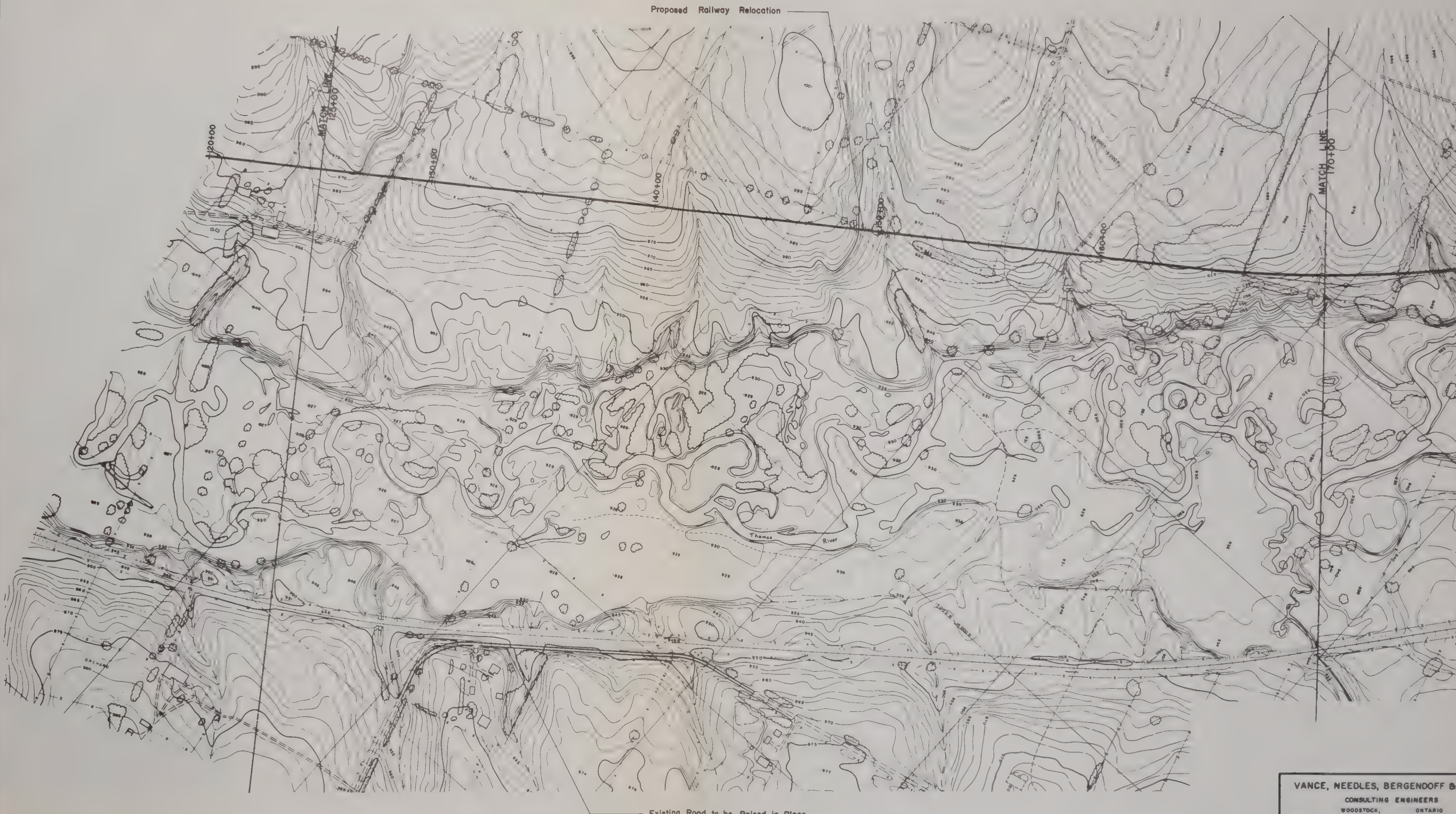


VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CANADIAN PACIFIC RAILWAY RELOCATION
PLAN
ALTERNATE 1 - HIGH DAM

SCALE 1" = 400'
DATE OCT. 15, 1961
DRAWING NO. 9

NO	REVISION MADE	BY	DATE



NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CANADIAN PACIFIC RAILWAY RELOCATION
PLAN
ALTERNATE I - HIGH DAM

SCALE: 1" = 400'
DATE: OCT. 15, 1967
DRAWING NO. 10

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CANADIAN PACIFIC RAILWAY RELOCATION
PLAN
ALTERNATE 1 - HIGH DAM

SCALE 1"=400'
DATE OCT. 15, 1961
DRAWING NO 11

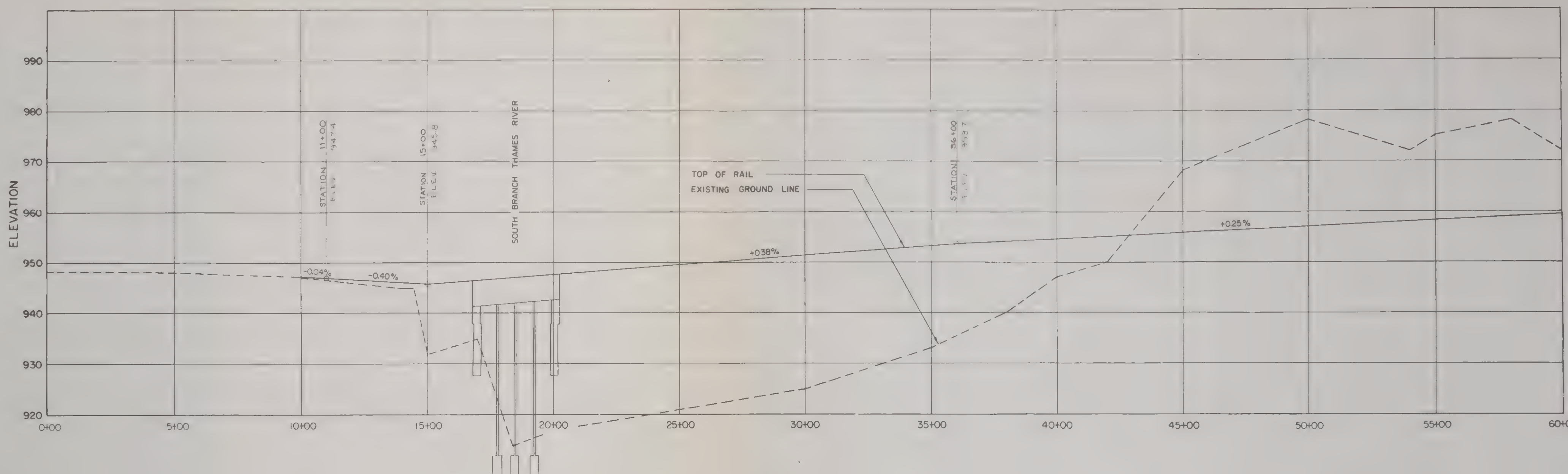
NO	REVISION MADE	BY	DATE

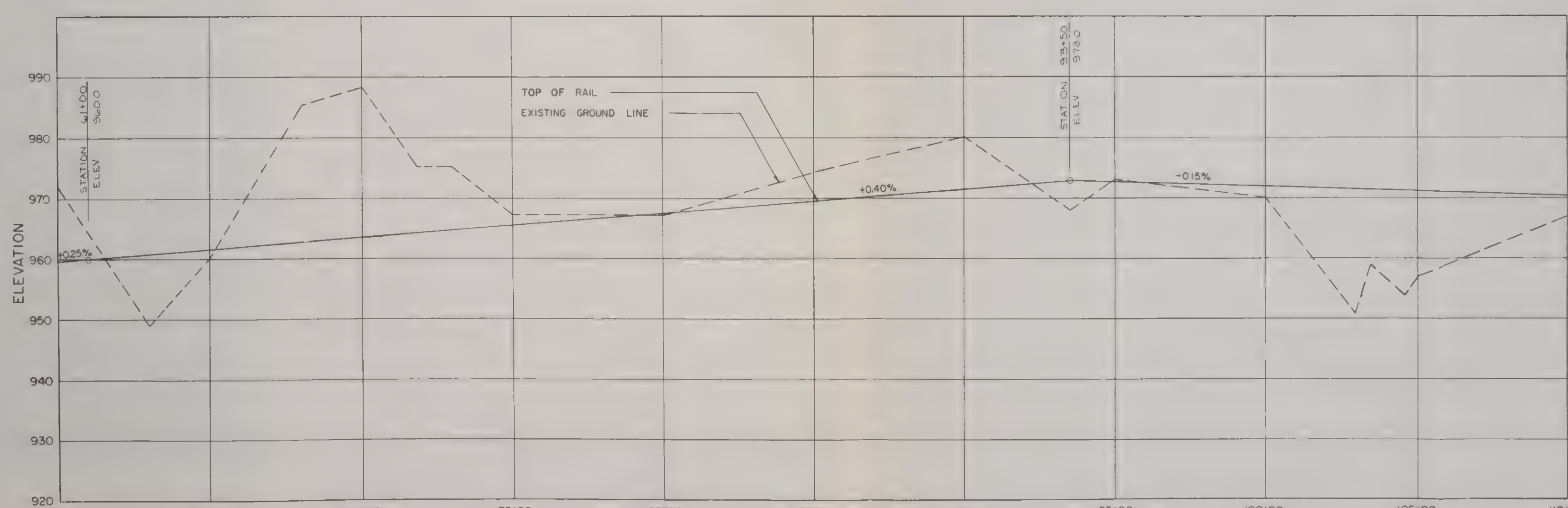
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CANADIAN PACIFIC RAILWAY RELOCATION
PLAN
ALTERNATE 1 - HIGH DAM

SCALE: 1" = 400'
DATE: OCT 15, 1961
DRAWING NO. 12



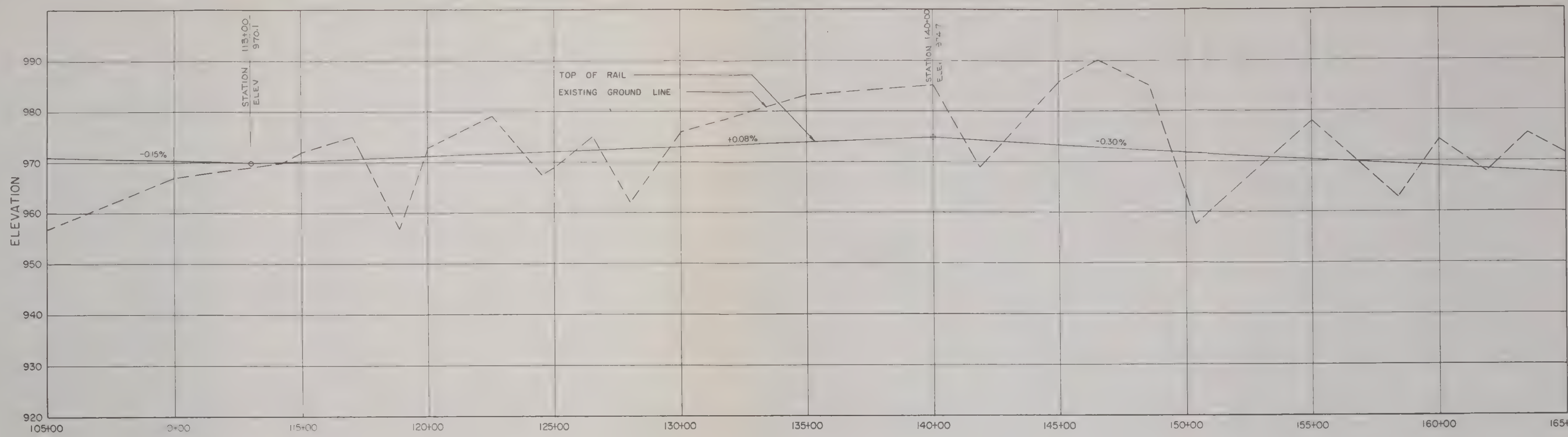


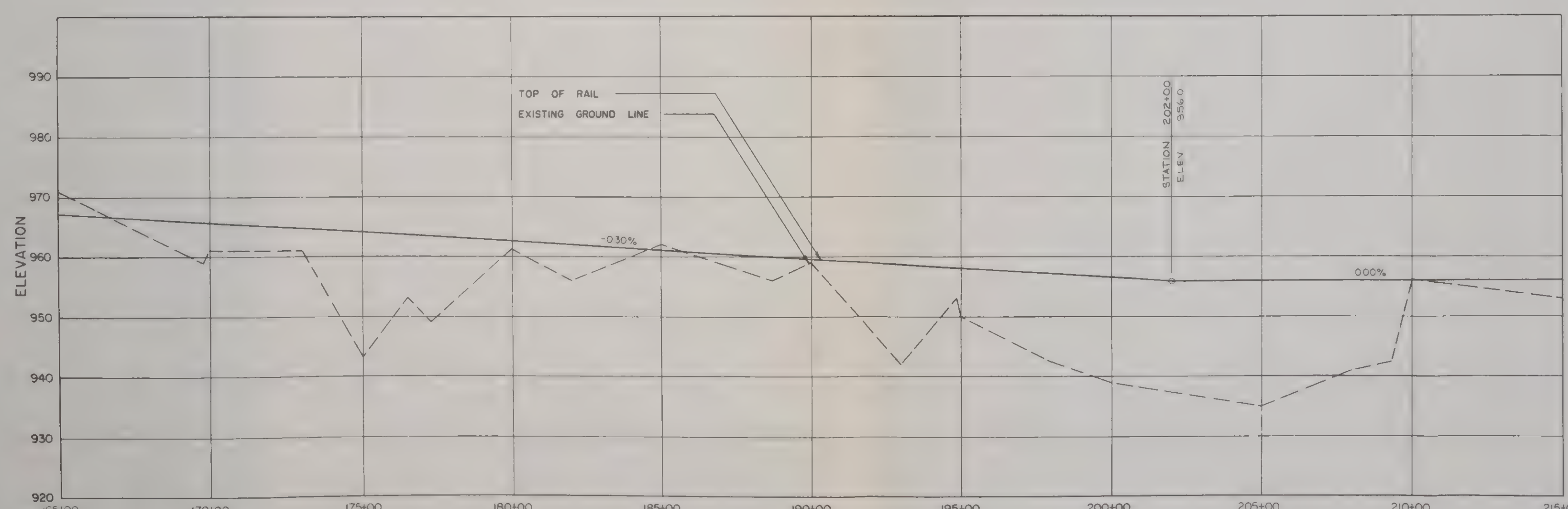
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CANADIAN PACIFIC RAILWAY RELOCATION
PROPOSED PROFILE
ALTERNATE I - HIGH DAM

SCALE Hor. 1" = 400' Vert. 1" = 20'
DATE OCT 15, 1961
DRAWING NO. 13





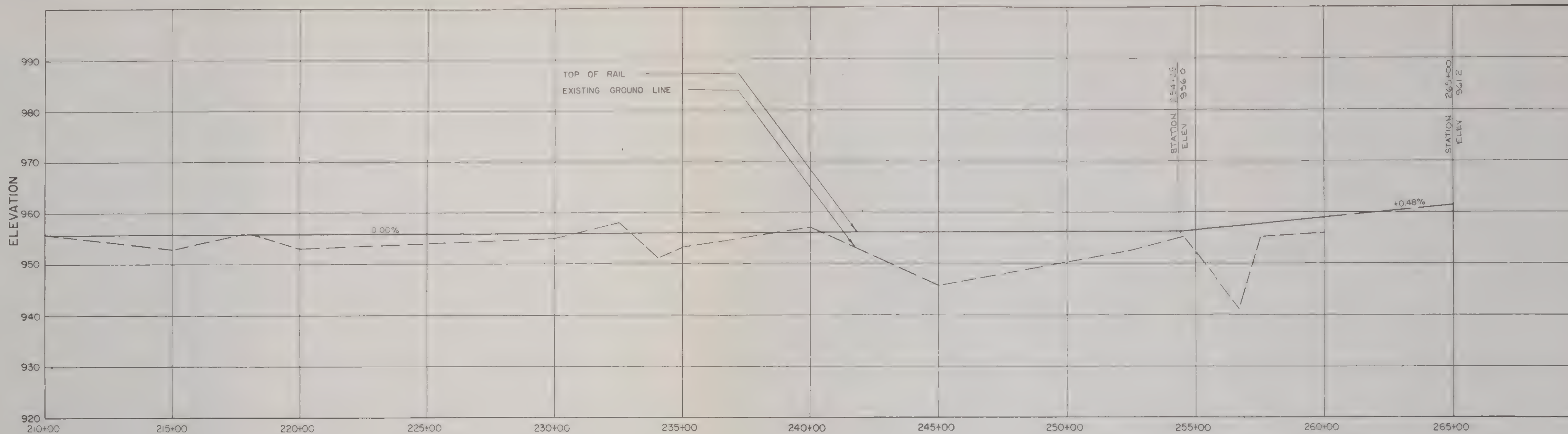
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CANADIAN PACIFIC RAILWAY RELOCATION
PROPOSED PROFILE
ALTERNATE I - HIGH DAM

SCALE Hor. 1" = 400' Vert. 1" = 20'
DATE OCT. 15, 1961

DRAWING NO. 14



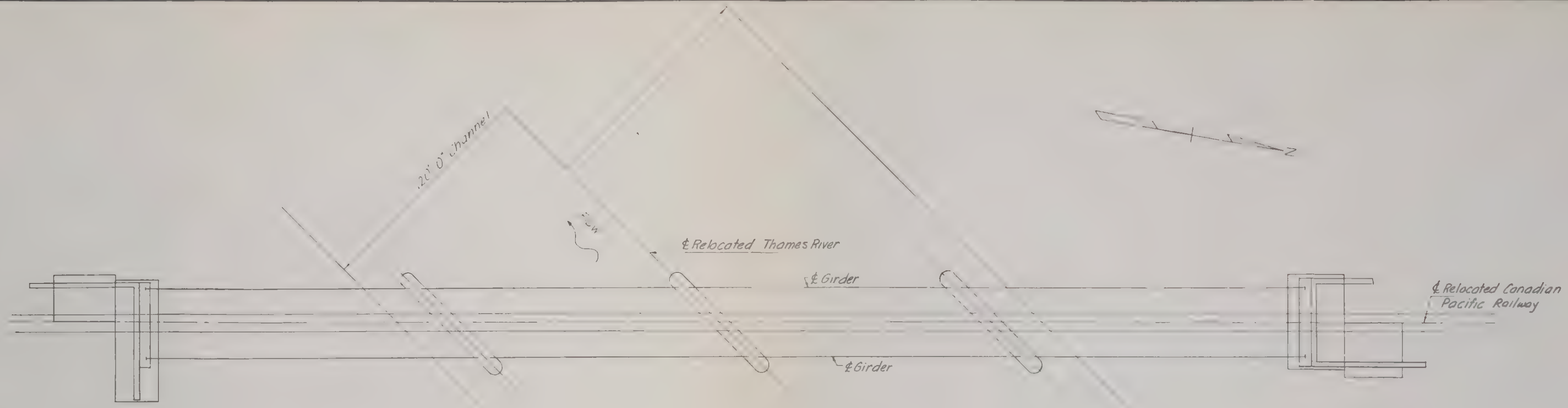
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

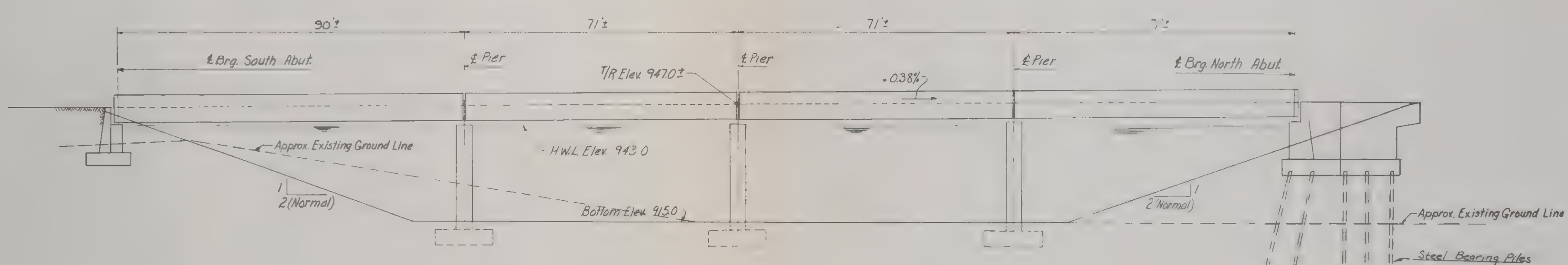
UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CANADIAN PACIFIC RAILWAY RELOCATION
PROPOSED PROFILE
ALTERNATE I - HIGH DAM

SCALE Hor. 1" = 400' Vert. 1" = 20'
DATE: OCT. 15, 1961

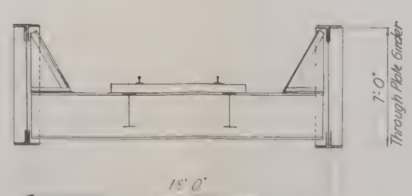
DRAWING NO. 15



PLAN
1" = 30'



ELEVATION
1" = 30'



TYPICAL CROSS SECTION
1/4" = 2'-0"

NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CANADIAN PACIFIC RAILWAY RELOCATION
PROPOSED BRIDGE OVER S. B. THAMES RIVER
ALTERNATE 1 – HIGH DAM

SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 16

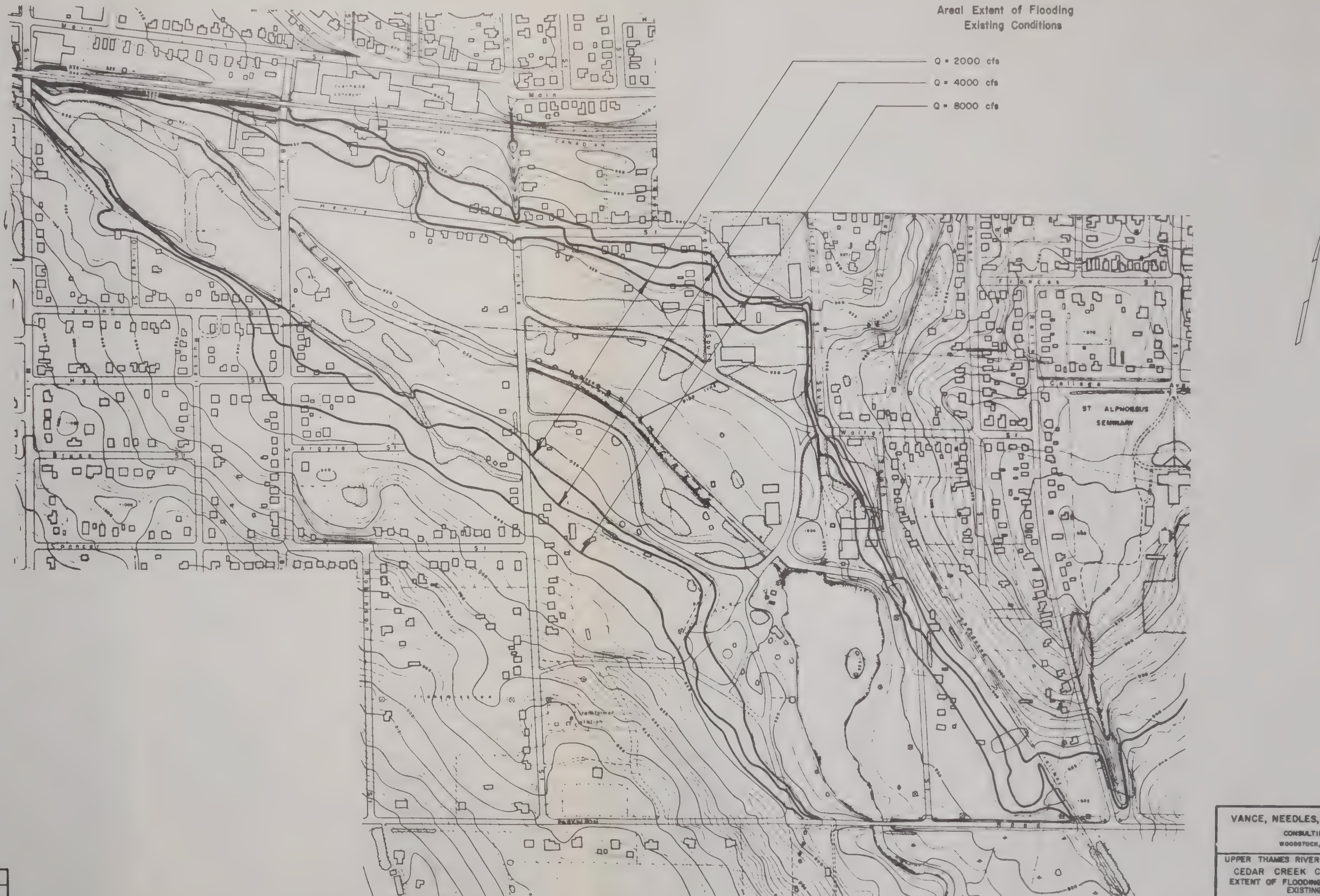
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK CHANNEL IMPROVEMENT
EXTENT OF FLOODING AT VARIOUS DISCHARGES
EXISTING CONDITIONS
S. B. THAMES RIVER TO MILL STREET
ALTERNATE 1 - HIGH DAM

SCALE: 1" = 400'
DATE: OCT. 15, 1961

DRAWING NO. 17



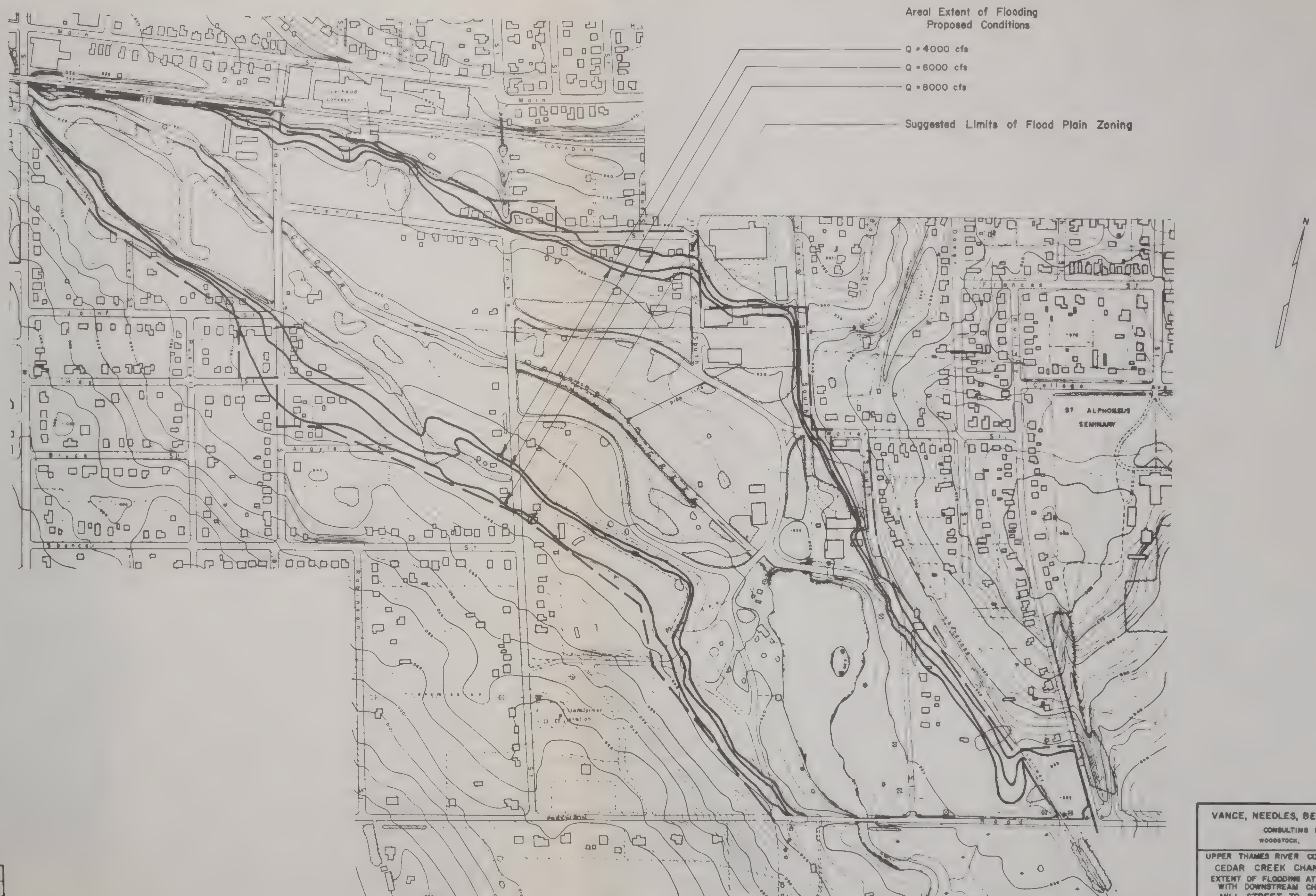
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK CHANNEL IMPROVEMENT
EXTENT OF FLOODING AT VARIOUS DISCHARGES
EXISTING CONDITIONS
MILL STREET TO PARKINSON ROAD
ALTERNATE I - HIGH DAM

SCALE: 1" = 400'
DATE: OCT 15 1961

DRAWING NO. 18



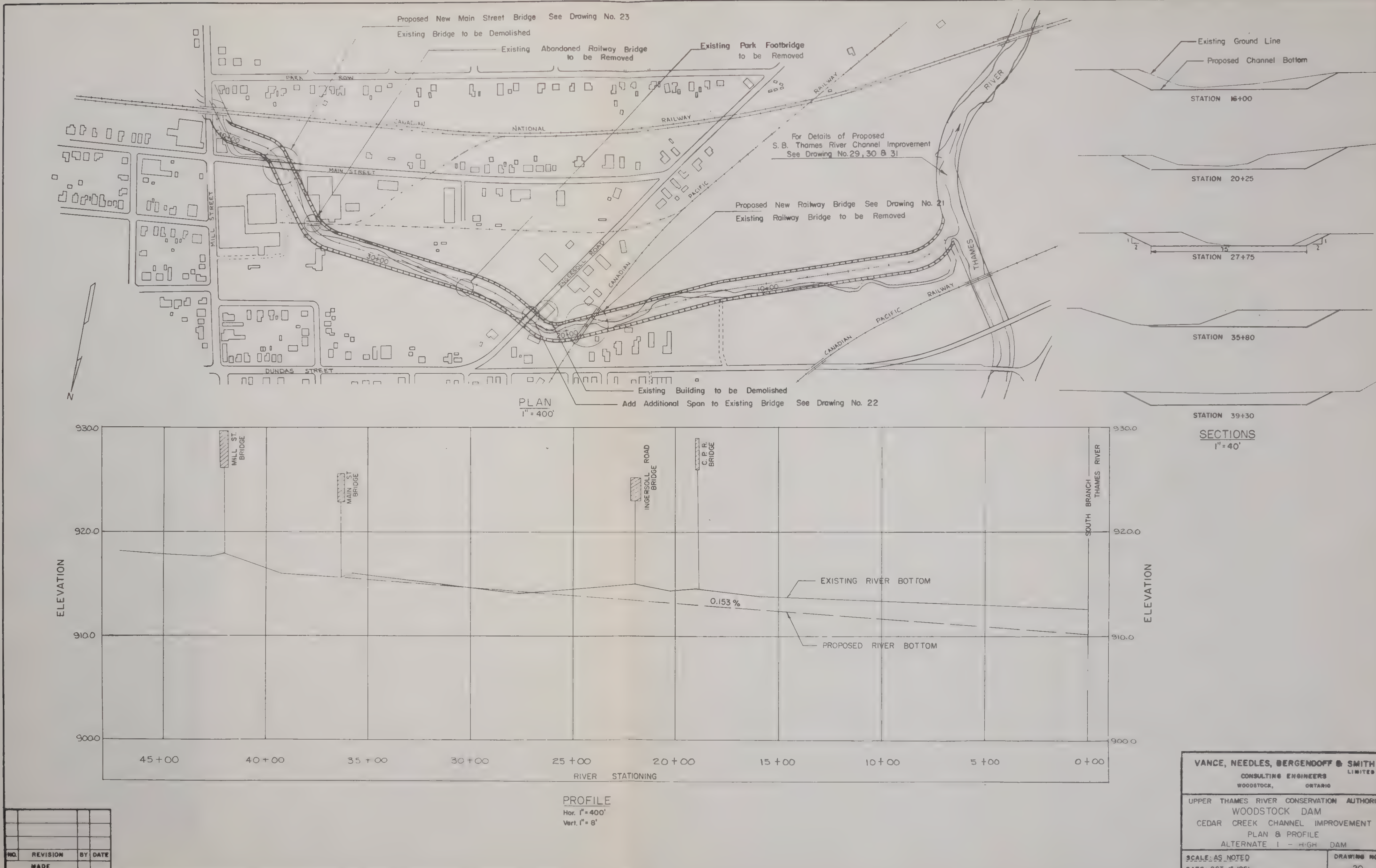
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

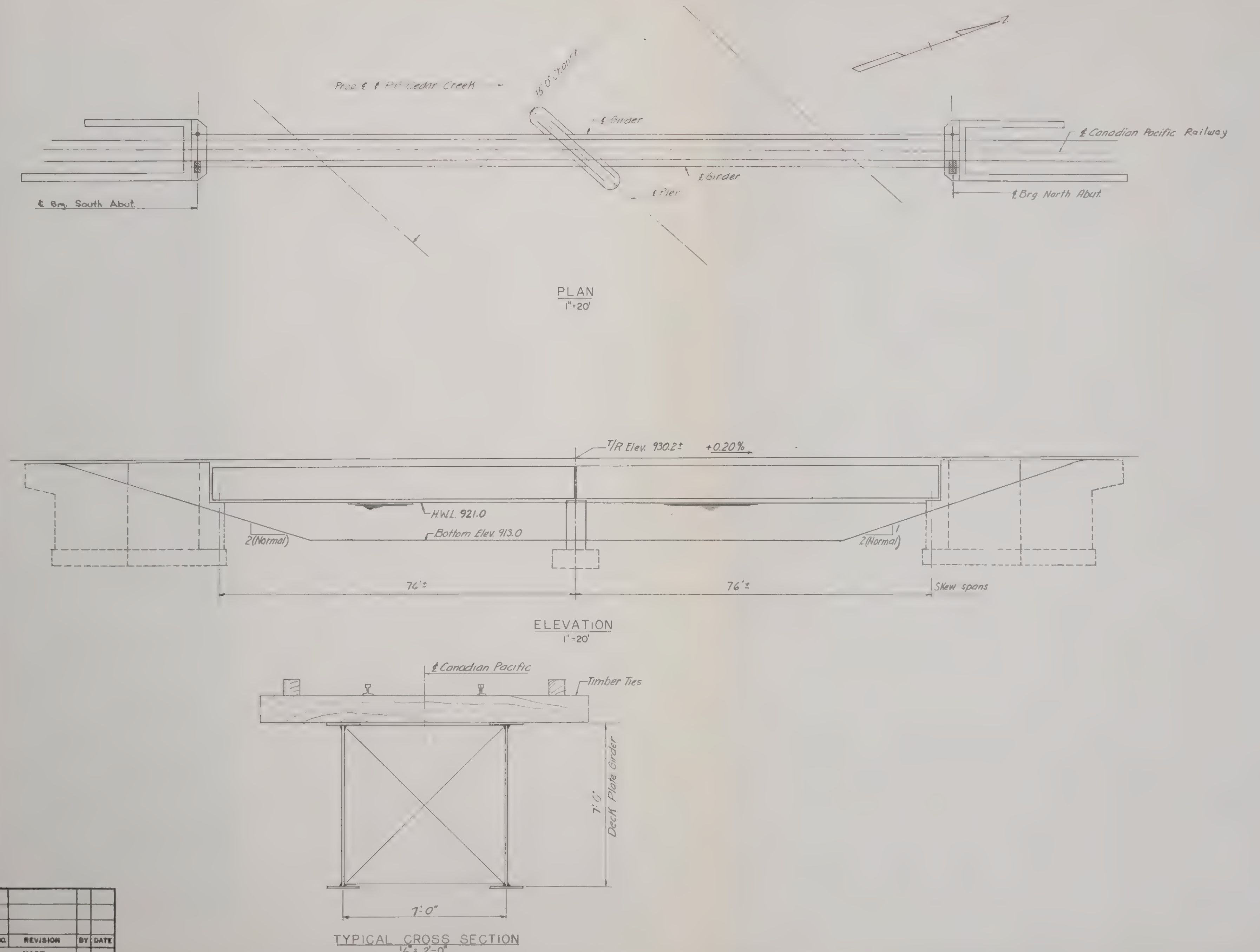
UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK CHANNEL IMPROVEMENT
EXTENT OF FLOODING AT VARIOUS DISCHARGES WITH DOWNSTREAM CHANNEL IMPROVEMENT
MILL STREET TO PARKINSON ROAD
ALTERNATE I - HIGH DAM

SCALE: 1" = 400'
DATE: OCT. 15, 1961

DRAWING NO. 19



Proposed New Main Street Bridge See Drawing No. 23
Existing Bridge to be Demolished
Existing Abandoned Railway Bridge to be Removed
Existing Park Footbridge to be Removed
PARK ROW
CANADIAN NATIONAL RAILWAY
MAIN STREET
MILL STREET
INGERSOLL ROAD
CANADIAN PACIFIC RAILWAY
DUNDAS STREET
RIVER
THAMES
For Details of Proposed S.B. Thames River Channel Improvement See Drawing No. 29, 30 & 31
Proposed New Railway Bridge See Drawing No. 21
Existing Railway Bridge to be Removed
Existing Building to be Demolished
Add Additional Span to Existing Bridge See Drawing No. 22
40+00
30+00
20+00
10+00
N
PLAN
1" = 400'
Existing Ground Line
Proposed Channel Bottom
STATION 16+00
STATION 20+25
STATION 27+75
75'
STATION 35+80
STATION 39+30
SECTIONS
1" = 40'
930.0
920.0
910.0
900.0
ELEVATION
MILL ST. BRIDGE
MAIN ST BRIDGE
INGERSOLL ROAD BRIDGE
C.P.R. BRIDGE
SOUTH BRANCH THAMES RIVER
EXISTING RIVER BOTTOM
0.153 %
PROPOSED RIVER BOTTOM
45+00
40+00
35+00
30+00
25+00
20+00
15+00
10+00
5+00
0+00
RIVER STATIONING
PROFILE
Hor. 1" = 400'
Vert. 1" = 8'
NO. REVISION MADE BY DATE
VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO
UPPER THAMES RIVER CONSERVATION AUTHORITY
WOODSTOCK DAM
CEDAR CREEK CHANNEL IMPROVEMENT
PLAN & PROFILE
ALTERNATE 1 - HIGH DAM
SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO.
20

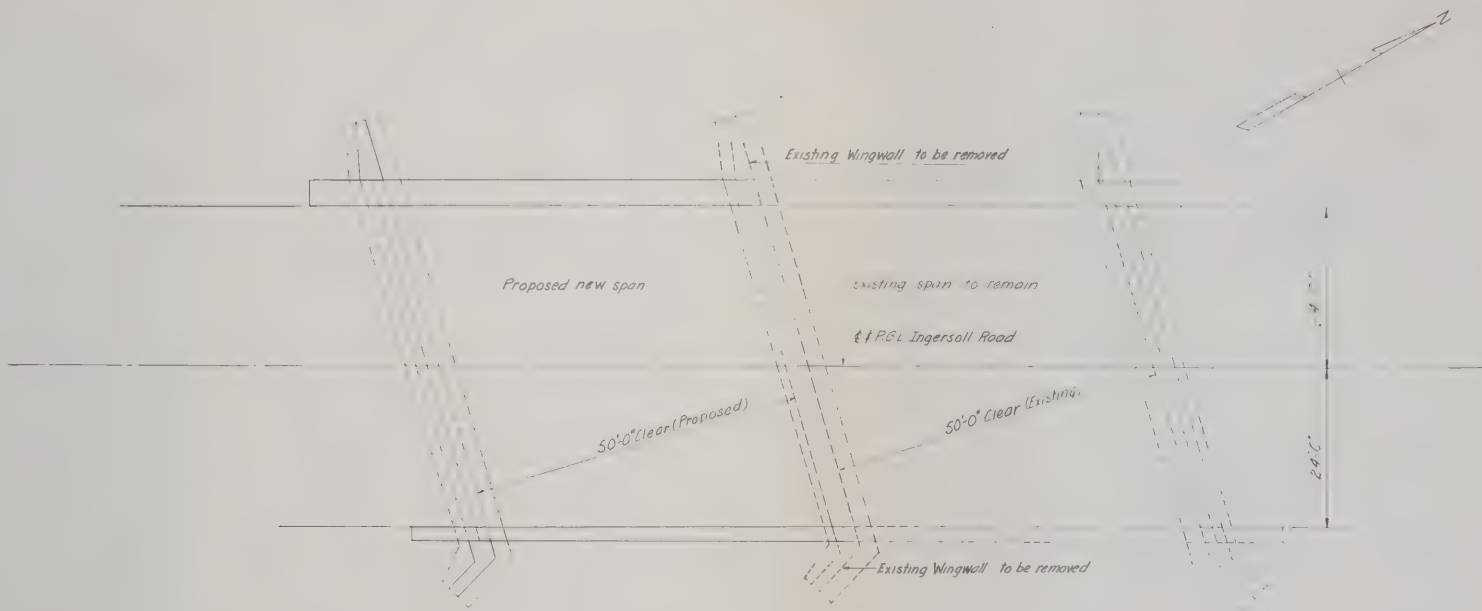


NO.	REVISION MADE	BY	DATE

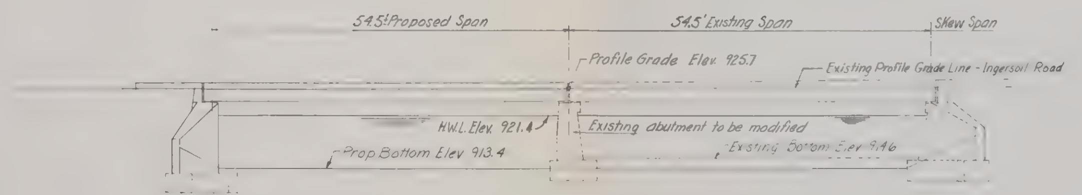
VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK CHANNEL IMPROVEMENT
C. P. R. BRIDGE OVER CEDAR CREEK
PLAN & ELEVATION
ALTERNATE 1 - HIGH DAM

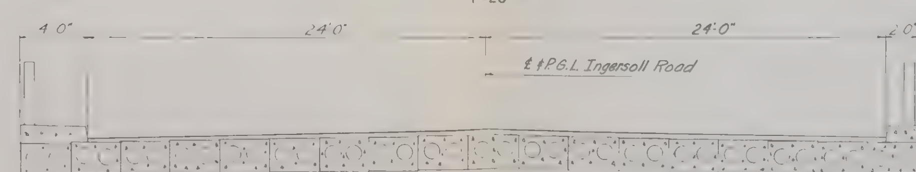
SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 21



PLAN
1" = 20'



ELEVATION
1" = 20'



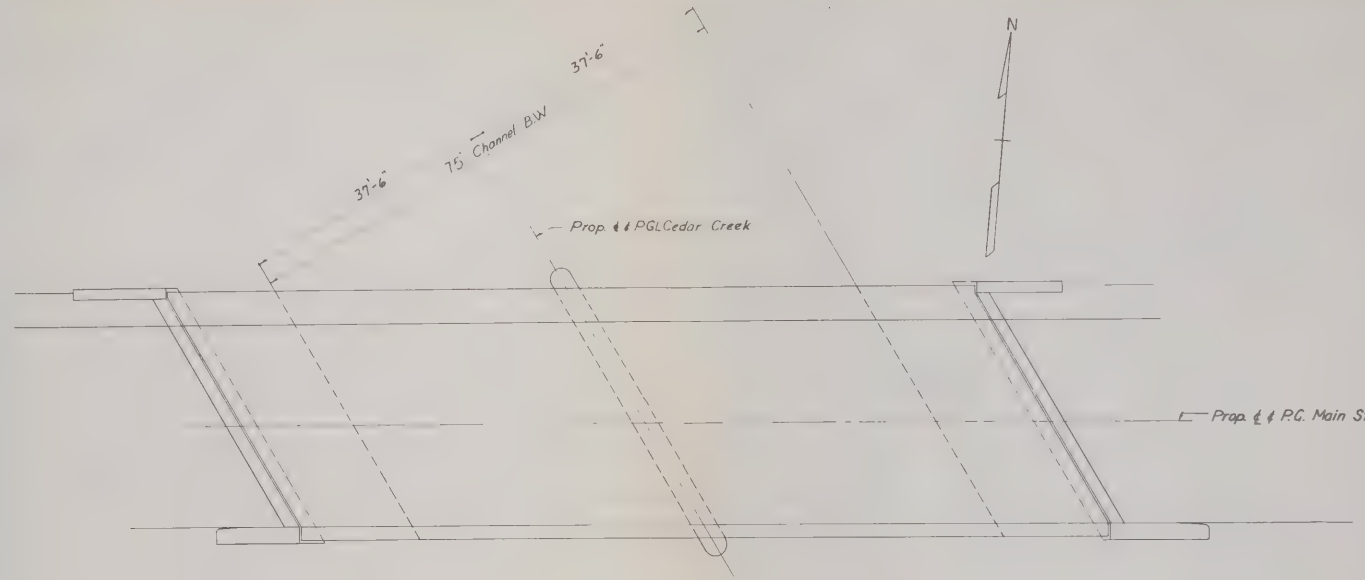
TYPICAL CROSS SECTION
1/4" = 2'-0"

NO	REVISION MADE	BY	DATE

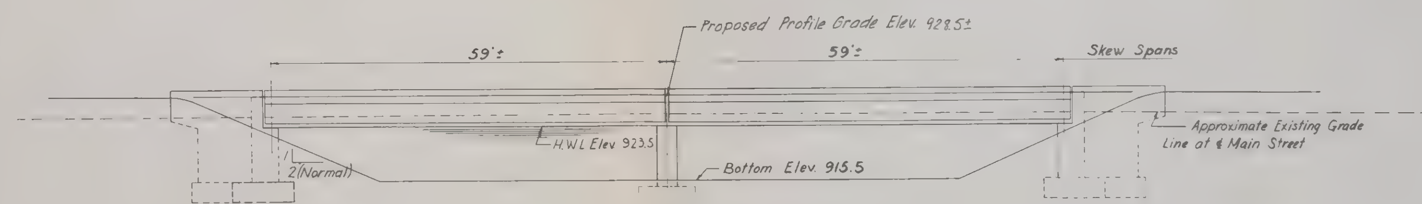
VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK CHANNEL IMPROVEMENT
INGERSOLL ROAD BRIDGE OVER CEDAR CREEK
PLAN & ELEVATION
ALTERNATE 1 - HIGH DAM

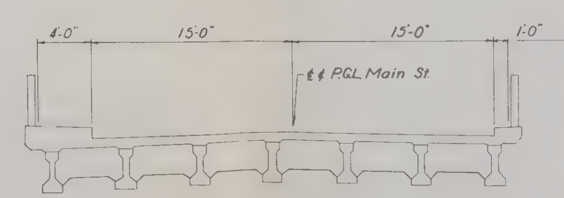
SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 22



PLAN
1" = 20'



ELEVATION
1" = 20'



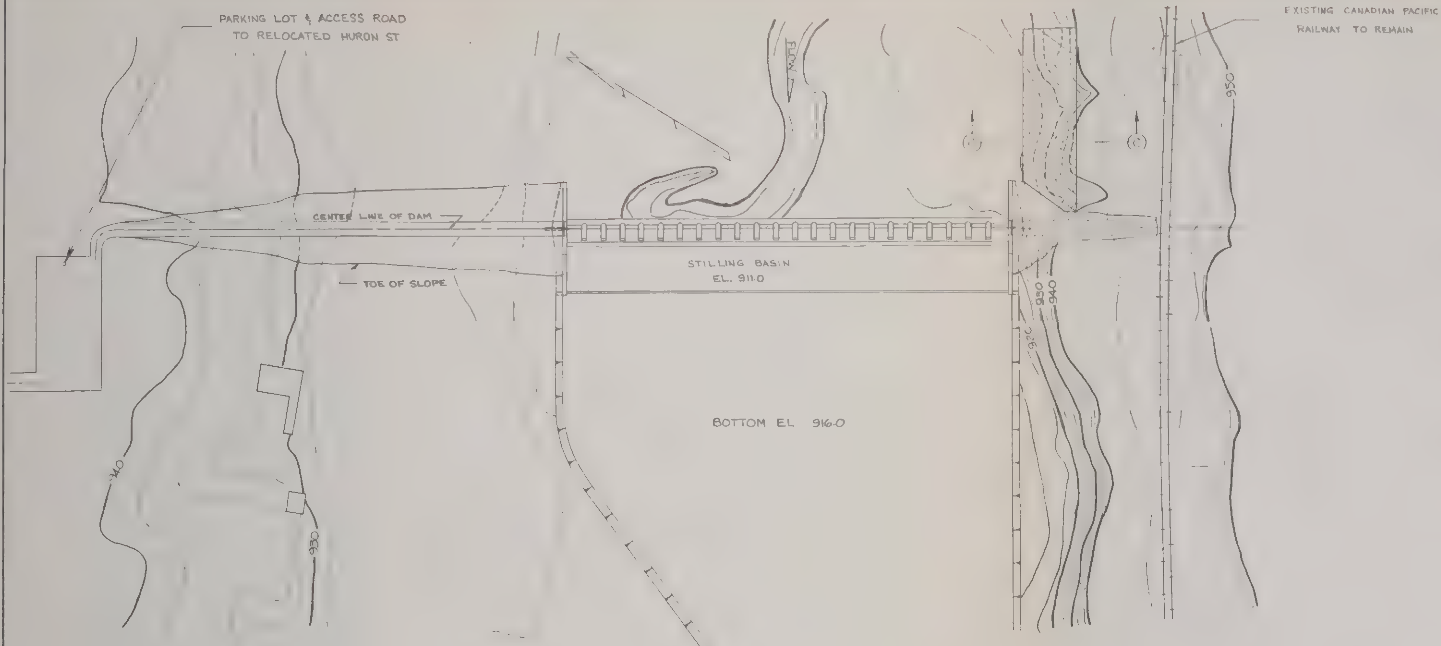
TYPICAL CROSS SECTION
1" = 10'-0"

NO	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

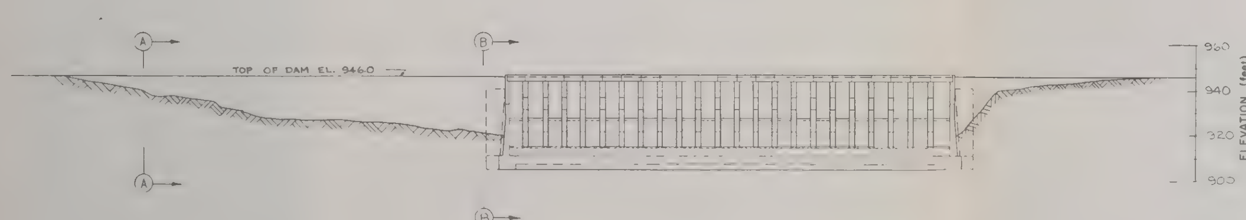
UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK CHANNEL IMPROVEMENT
MAIN STREET BRIDGE OVER CEDAR CREEK
PLAN & ELEVATION
ALTERNATE 1 - HIGH DAM

SCALE AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 23



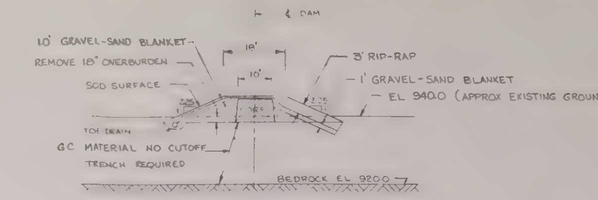
PLAN

1" = 200'

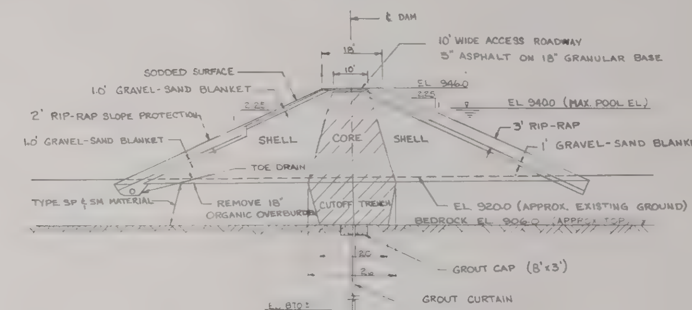


ELEVATION

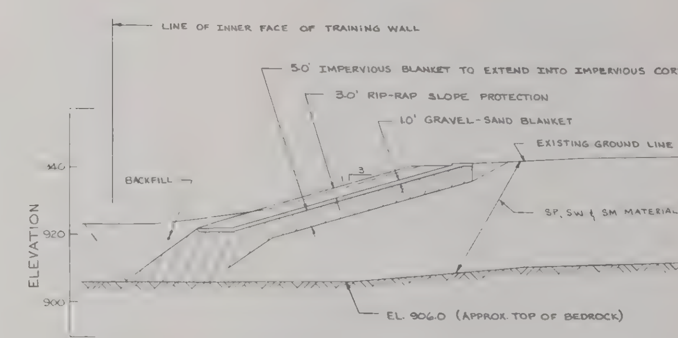
HORIZONTAL 1" = 200'
VERTICAL 1" = 60'



SECTION A-A



SECTION B-B



SECTION C-C

EMBANKMENT SECTIONS

1" = 40

NOTES:

1. CORE & CUTOFF TRENCH BACKFILL SHALL BE TYPE CG MATERIAL.
2. SHELL SHALL BE TYPES SP, SW, GP OR GW MATERIAL.
3. GRAVEL-SAND BLANKET SHALL BE TYPE GW MATERIAL.

NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY

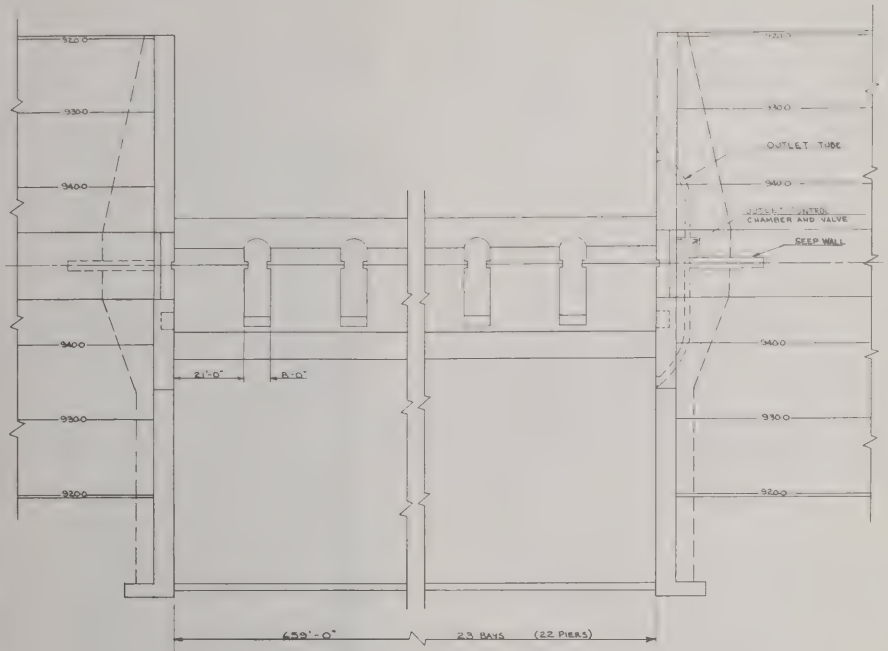
WOODSTOCK DAM

PLAN, ELEVATION & SECTIONS

ALTERNATE 2 - LOW DAMS

SCALE AS NOTED
DATE OCT 15, 1961

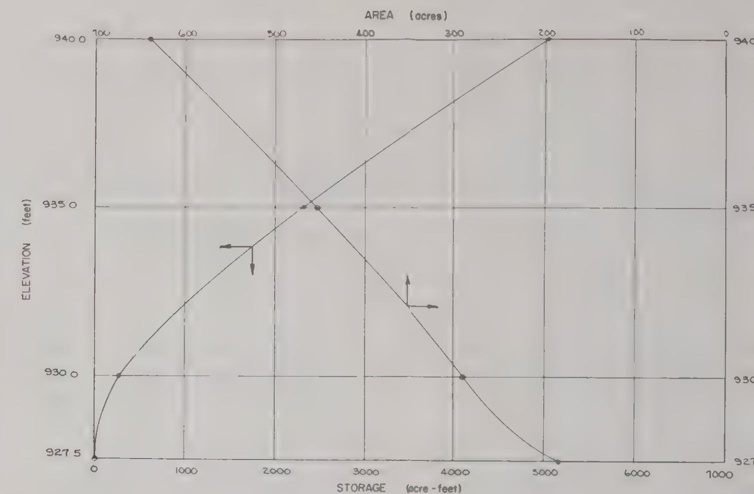
DRAWING NO 24



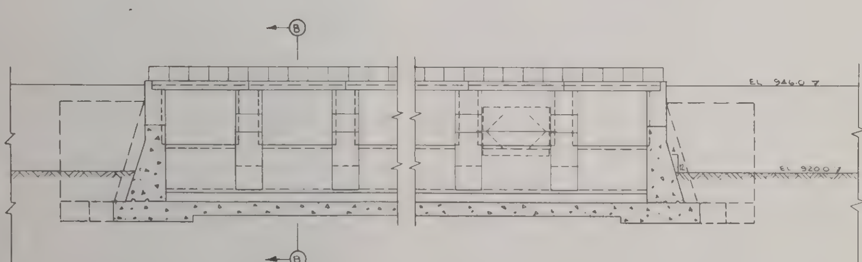
PLAN

(BRIDGE NOT SHOWN)

1" = 40'



AREA STORAGE CURVES

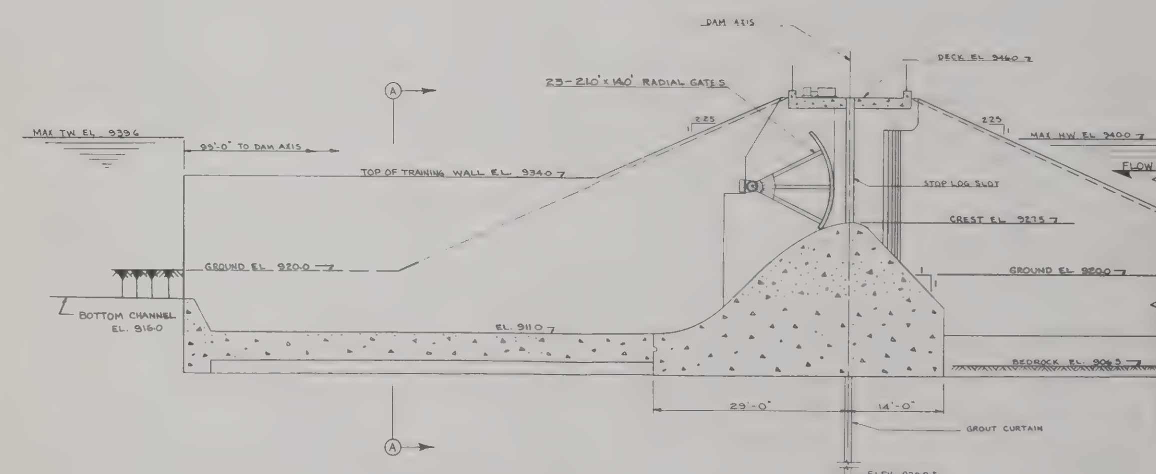


ELEVATION

SECTION A-A

(ONE GATE SHOWN)

1" = 40'



SECTION B-B

1" = 20'

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED

CONSULTING ENGINEERS

WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY

WOODSTOCK DAM

DETAILS OF SPILLWAY SECTION

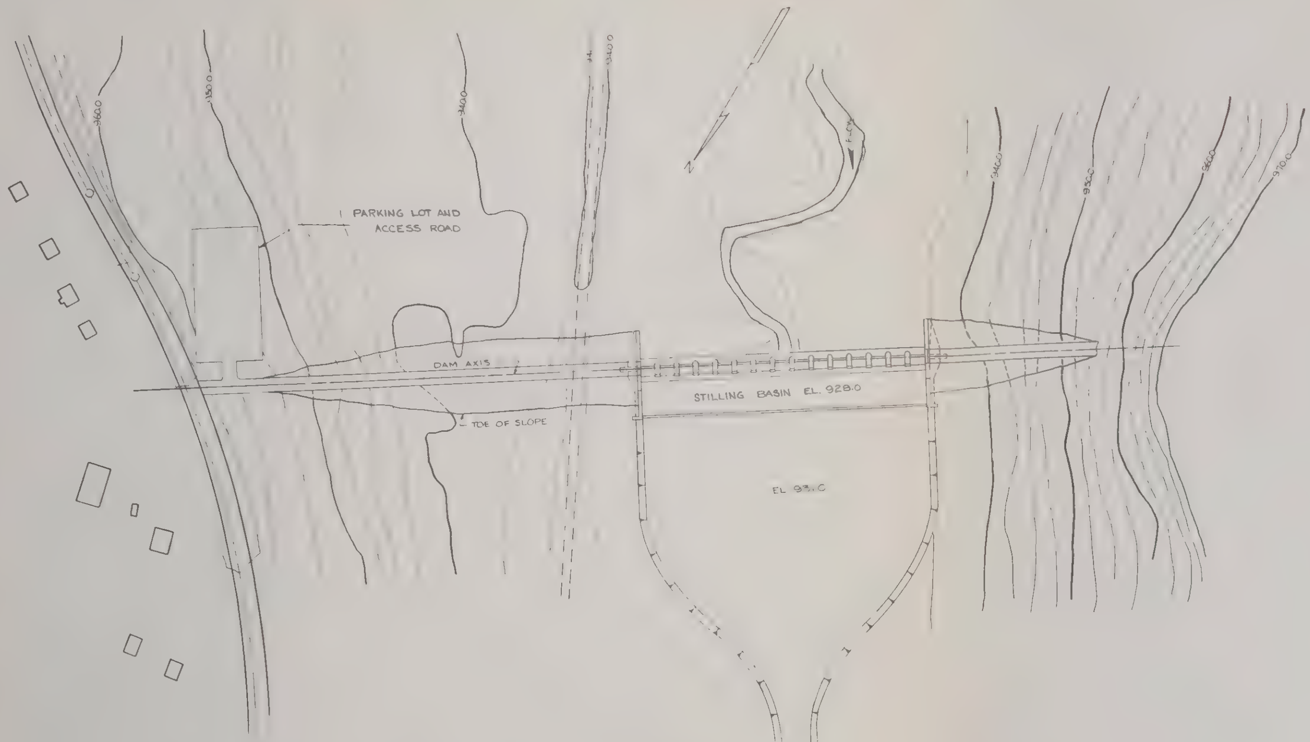
ALTERNATE 2 - LOW DAMS

SCALE: AS NOTED

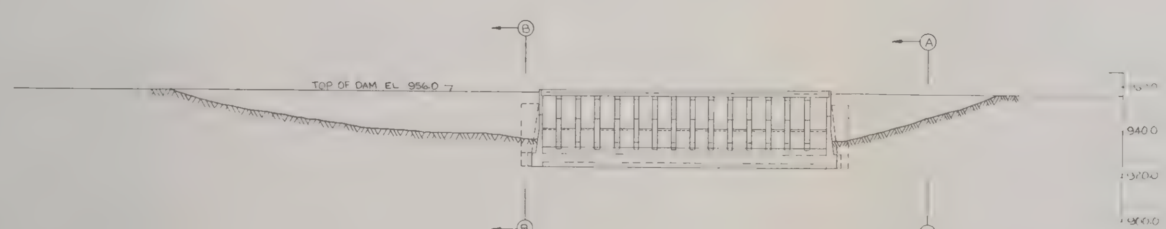
DATE: OCT. 15, 1961

DRAWING NO. 25

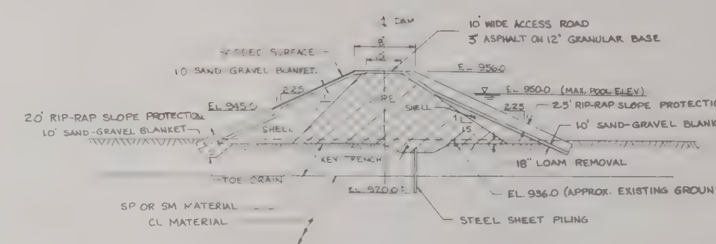
NO.	REVISION MADE	BY	DATE



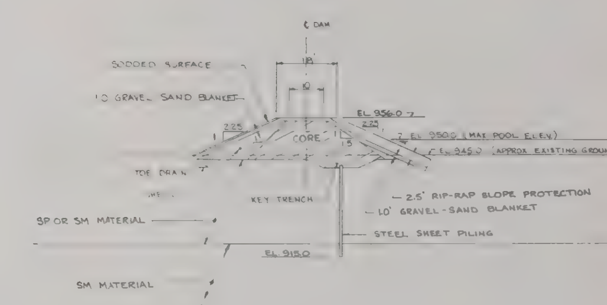
PLAN
1"=200'



ELEVATION
HORIZONTAL 1"=200'
VERTICAL 1"=60'



SECTION B-B



SECTION A-A

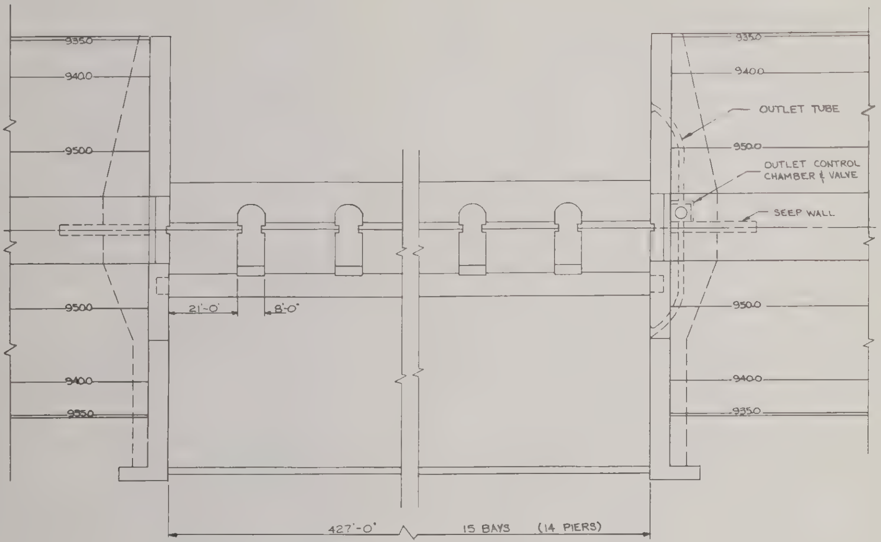
EMBANKMENT SECTIONS
1"=40'

NO.	REVISION MADE	BY	DATE

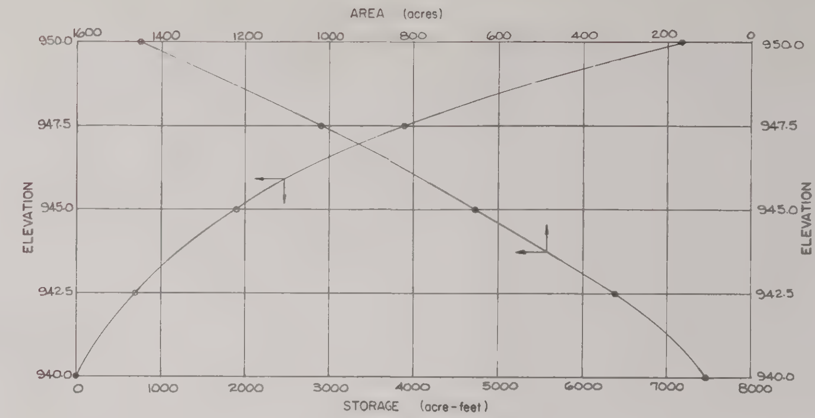
VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK DAM
PLAN, ELEVATION & SECTIONS
ALTERNATE 2 - LOW DAMS

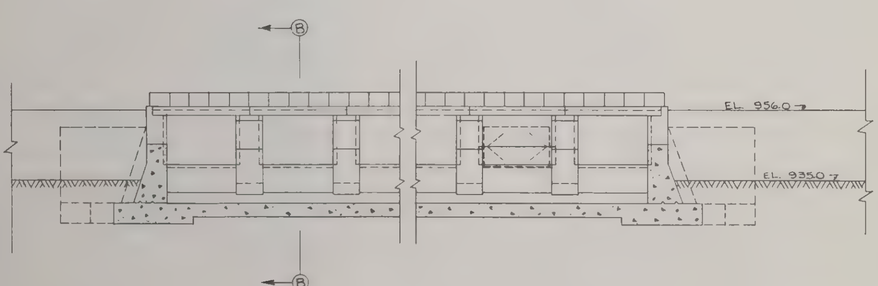
SCALE AS NOTED
DATE OCT. 15, 1961
DRAWING NO. 26



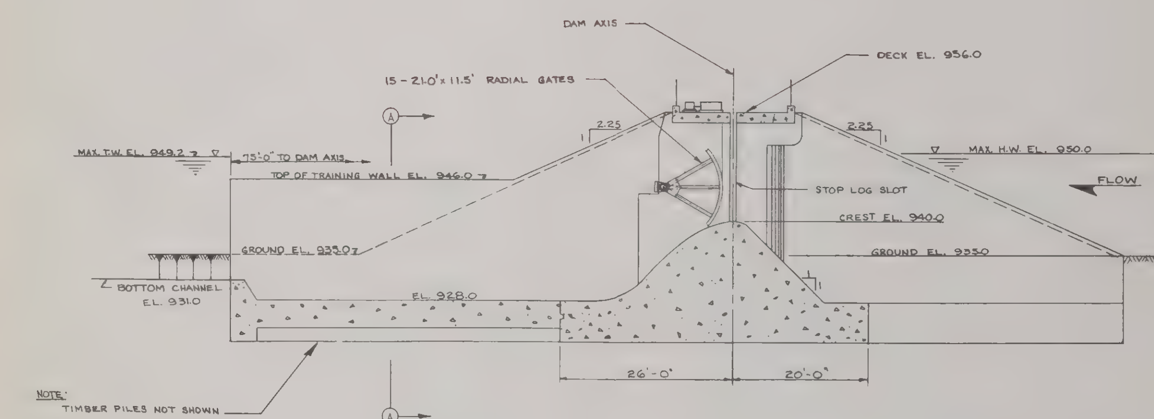
PLAN
(BRIDGE NOT SHOWN)
1" = 40'



AREA STORAGE CURVES



ELEVATION
SECTION A-A
(ONE GATE SHOWN)
1" = 40'



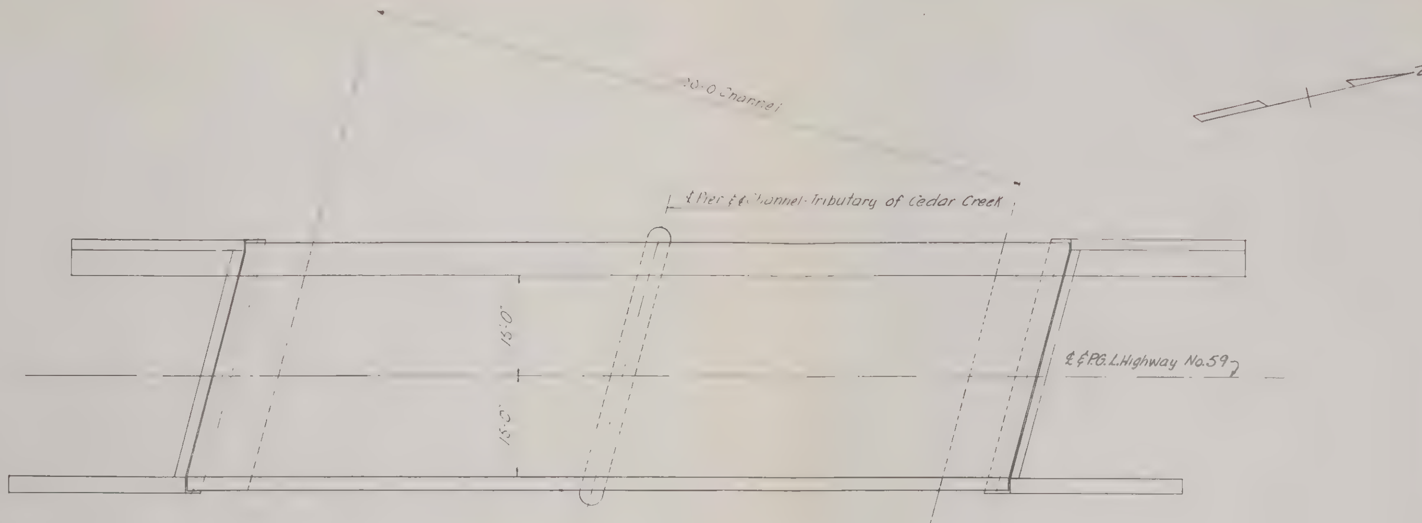
SECTION B-B
1" = 20'

NO.	REVISION MADE	BY	DATE

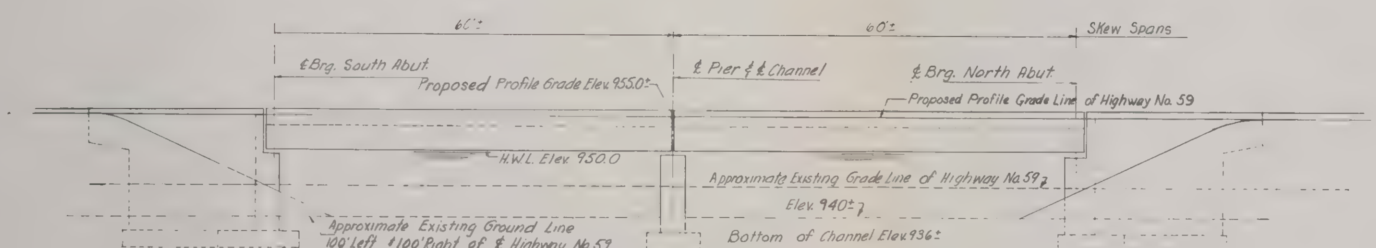
VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK DAM
DETAILS OF SPILLWAY SECTION
ALTERNATE 2- LOW DAMS

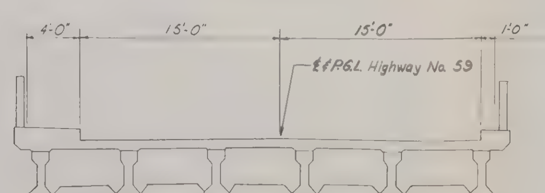
SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 27



PLAN
1"=20'



ELEVATION
1"=20'



TYPICAL CROSS SECTION
1" = 10'-0"

NO	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
CEDAR CREEK DAM
HIGHWAY NO. 59 BRIDGE OVER CEDAR CREEK
PLAN & ELEVATION
ALTERNATE 2 – LOW DAMS

SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 28

PROFILE
Hor. 1" = 200'
Vert. 1" = 8'

NO	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
S. B. THAMES RIVER
WOODSTOCK CHANNEL IMPROVEMENT
PLAN & PROFILE

SCALE: AS NOTED
DATE: OCT. 15, 1961
DRAWING NO. 29

NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
S. B. THAMES RIVER
WOODSTOCK CHANNEL IMPROVEMENT
PLAN & PROFILE

SCALE AS NOTED
DATE: OCT. 15, 1961

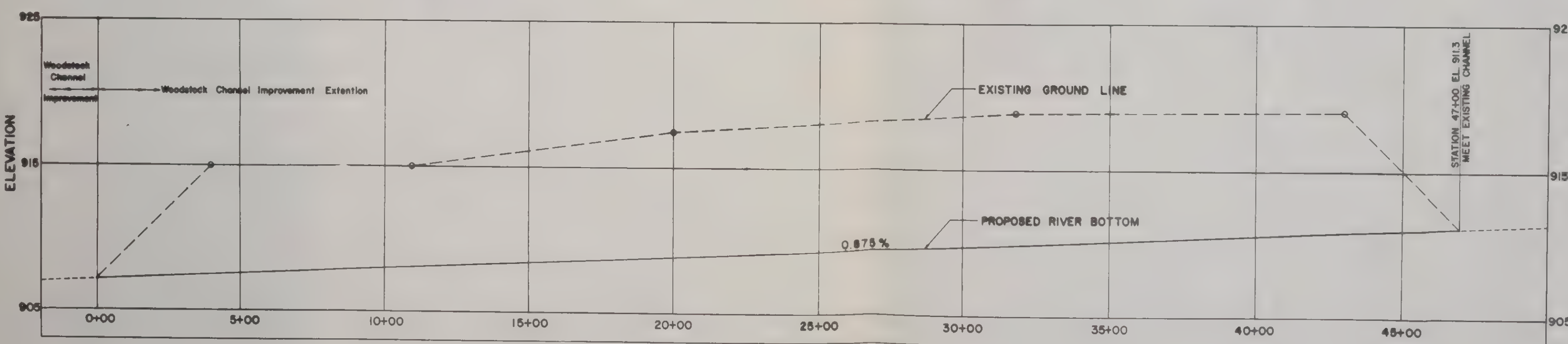
DRAWING NO. 30

ALTERNATE 1 - HIGH DAM
STATION 32+00

ALTERNATE 2 - LOW DAM
STATION 11+00

TYPICAL CROSS SECTIONS

PLAN
1"=400'



PROFILE
Hor. 1"=400'
Vert. 1"=8'

NO.	REVISION MADE	BY	DATE

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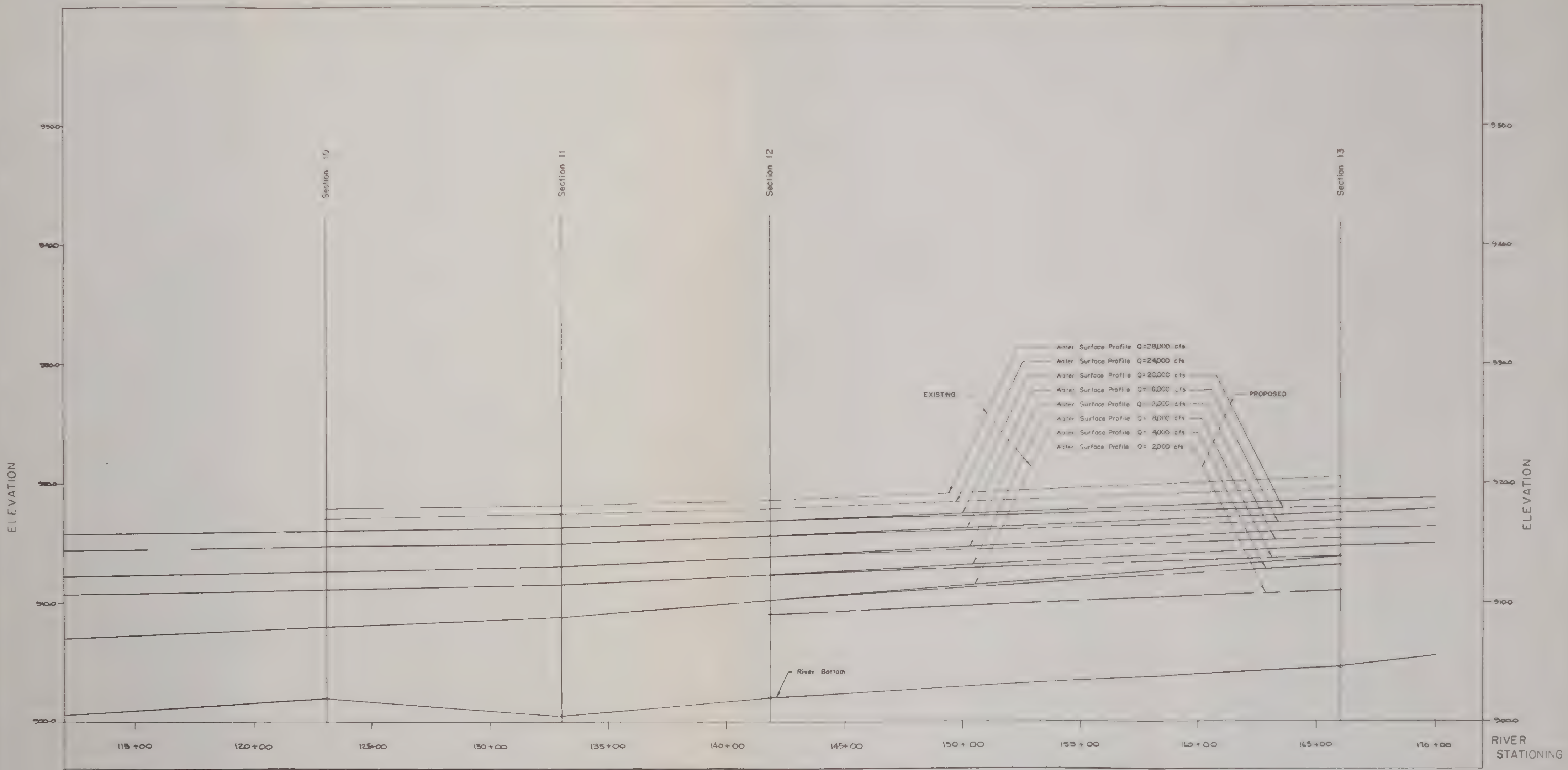
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

UPPER THAMES RIVER CONSERVATION AUTHORITY
S. B. THAMES RIVER
WOODSTOCK CHANNEL IMPROVEMENT EXTENSION
PLAN & PROFILE

SCALE AS NOTED
DATE OCT. 15, 1961
DRAWING NO. 31



PROFILE

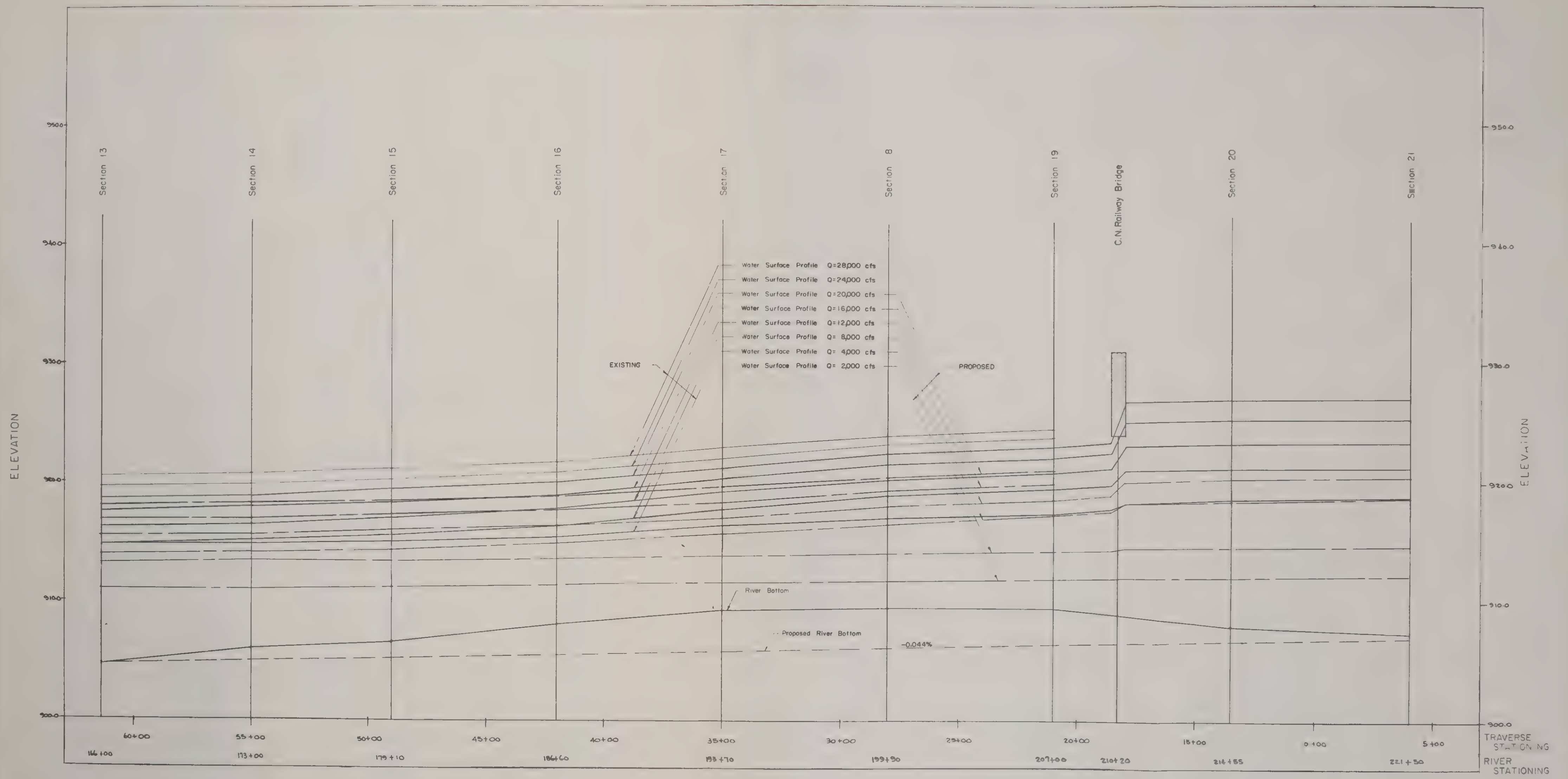
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

COMPUTED WATER SURFACE PROFILES
EXISTING AND PROPOSED CONDITIONS
S. B. THAMES RIVER

SCALE HOR. 1" = 400' VERT. 1" = 8'
DATE OCT. 15, 1961

DRAWING NO 32



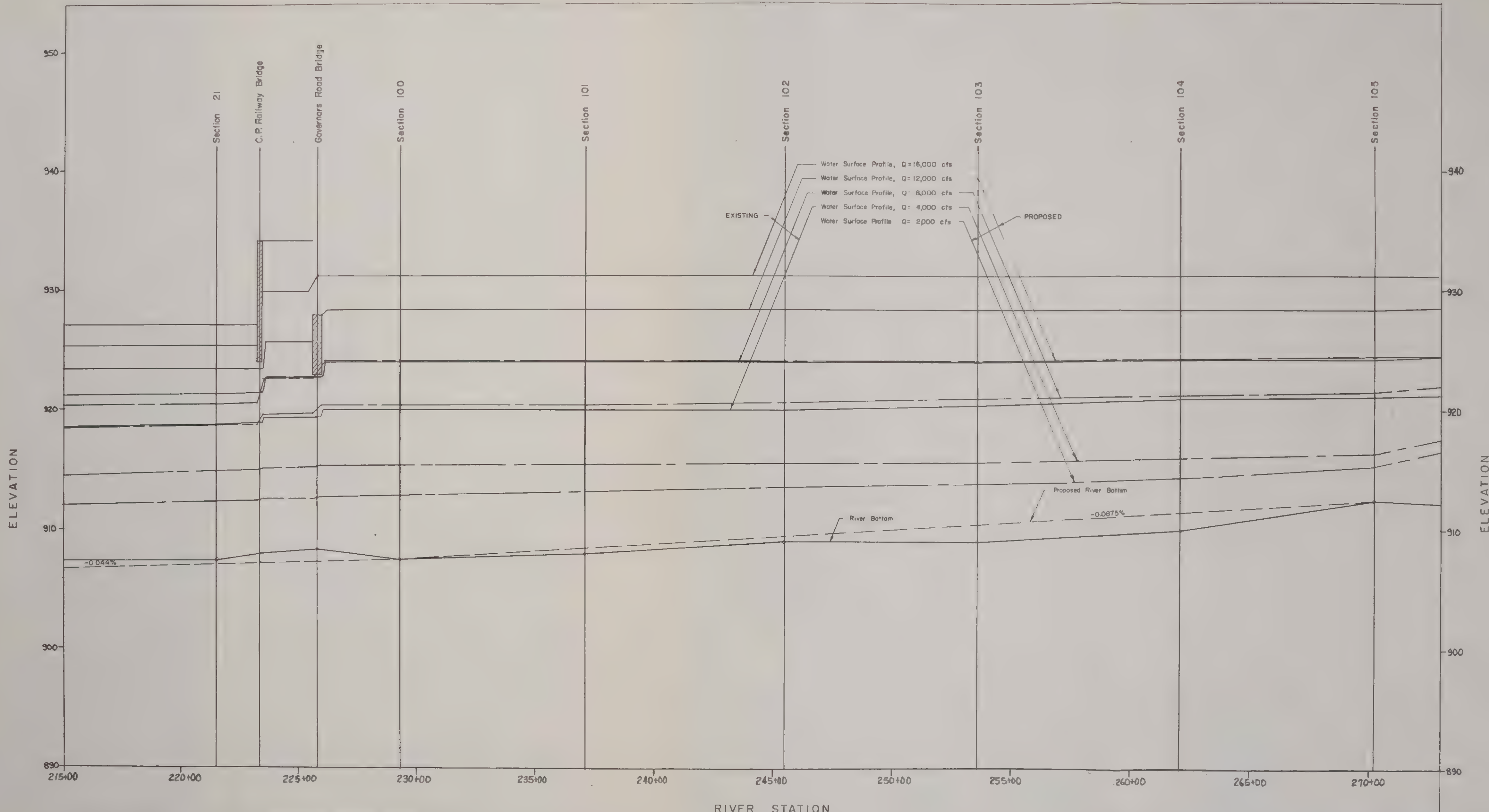
PROFILE

NO.	REVISION	BY	DATE
	MADE		

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

COMPUTED WATER SURFACE PROFILES
EXISTING AND PROPOSED CONDITIONS
S. B. THAMES RIVER

SCALE: HOR. 1"=400' VERT. 1"=8'
DATE: OCT. 15, 1961
DRAWING NO. 33



PROFILE

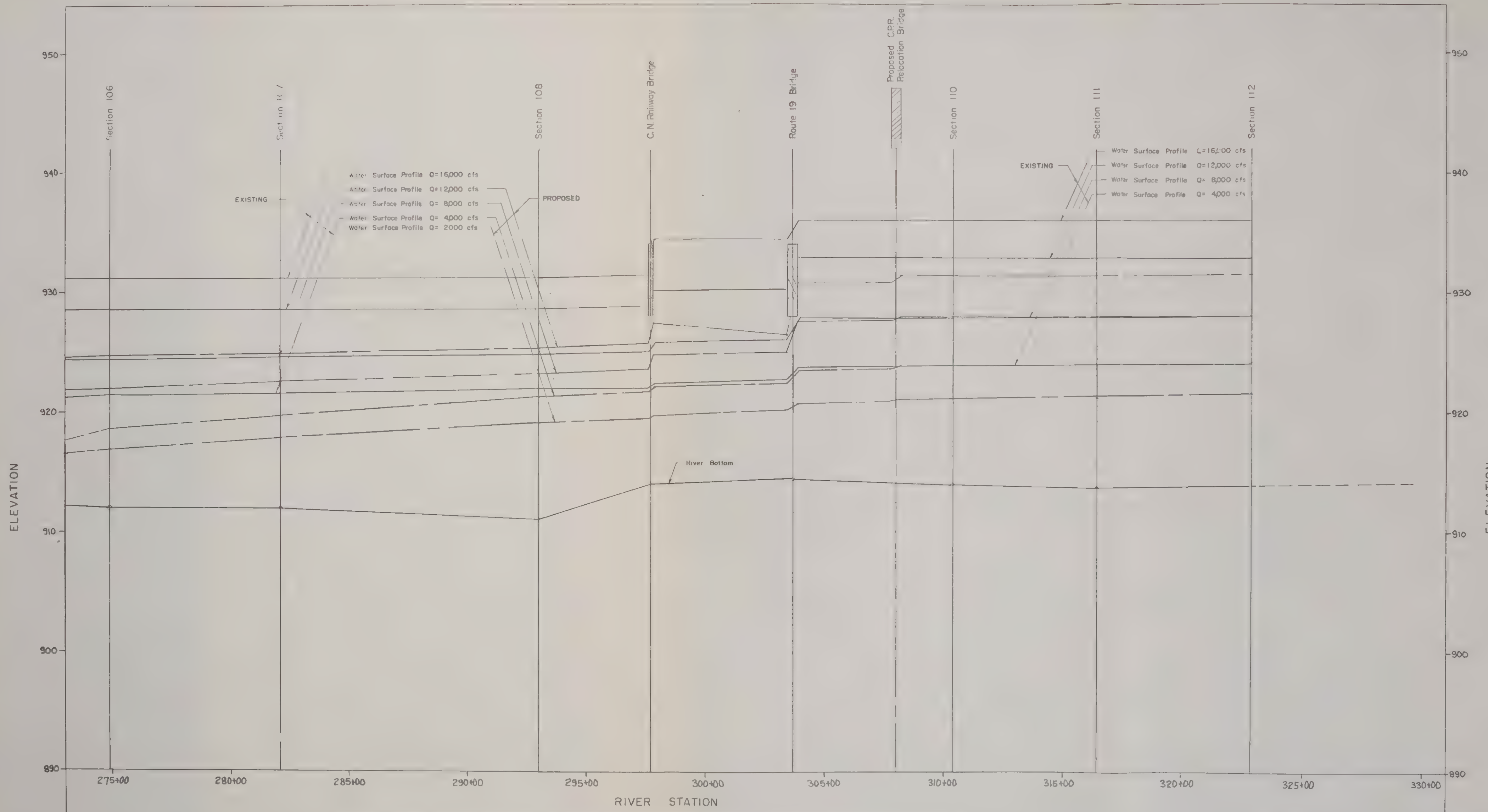
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

COMPUTED WATER SURFACE PROFILES
EXISTING AND PROPOSED CONDITIONS
S. B. THAMES RIVER

SCALE: HOR. 1"=400' VERT. 1"=8'
DATE: OCT. 15, 1961

DRAWING NO. 34



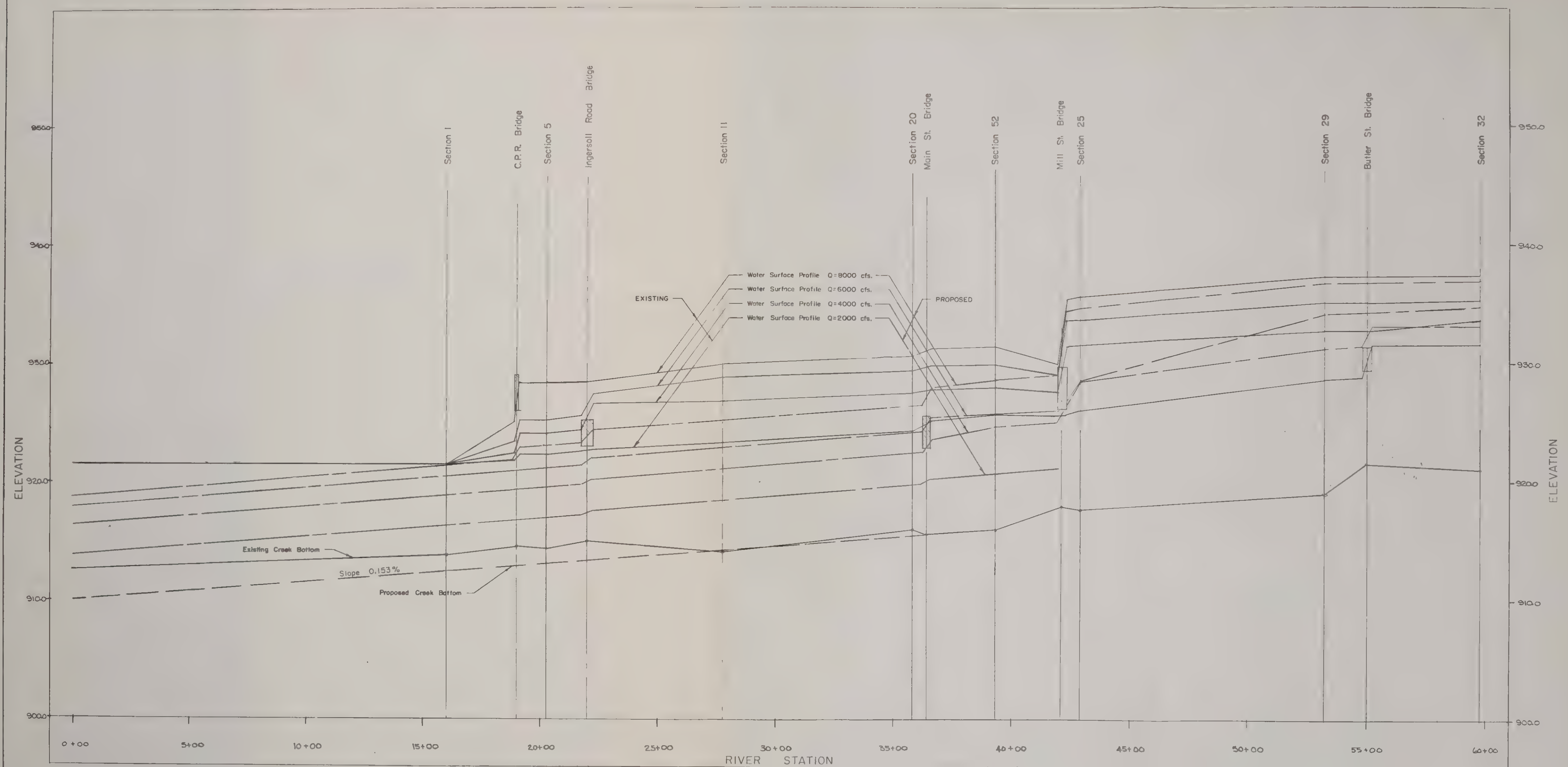
PROFILE

NO	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

COMPUTED WATER SURFACE PROFILES
EXISTING AND PROPOSED CONDITIONS
S. B. THAMES RIVER

SCALE: HOR. 1" = 400' VERT. 1" = 8'
DATE: OCT. 15, 1961
DRAWING NO 35



PROFILE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

COMPUTED WATER SURFACE PROFILES

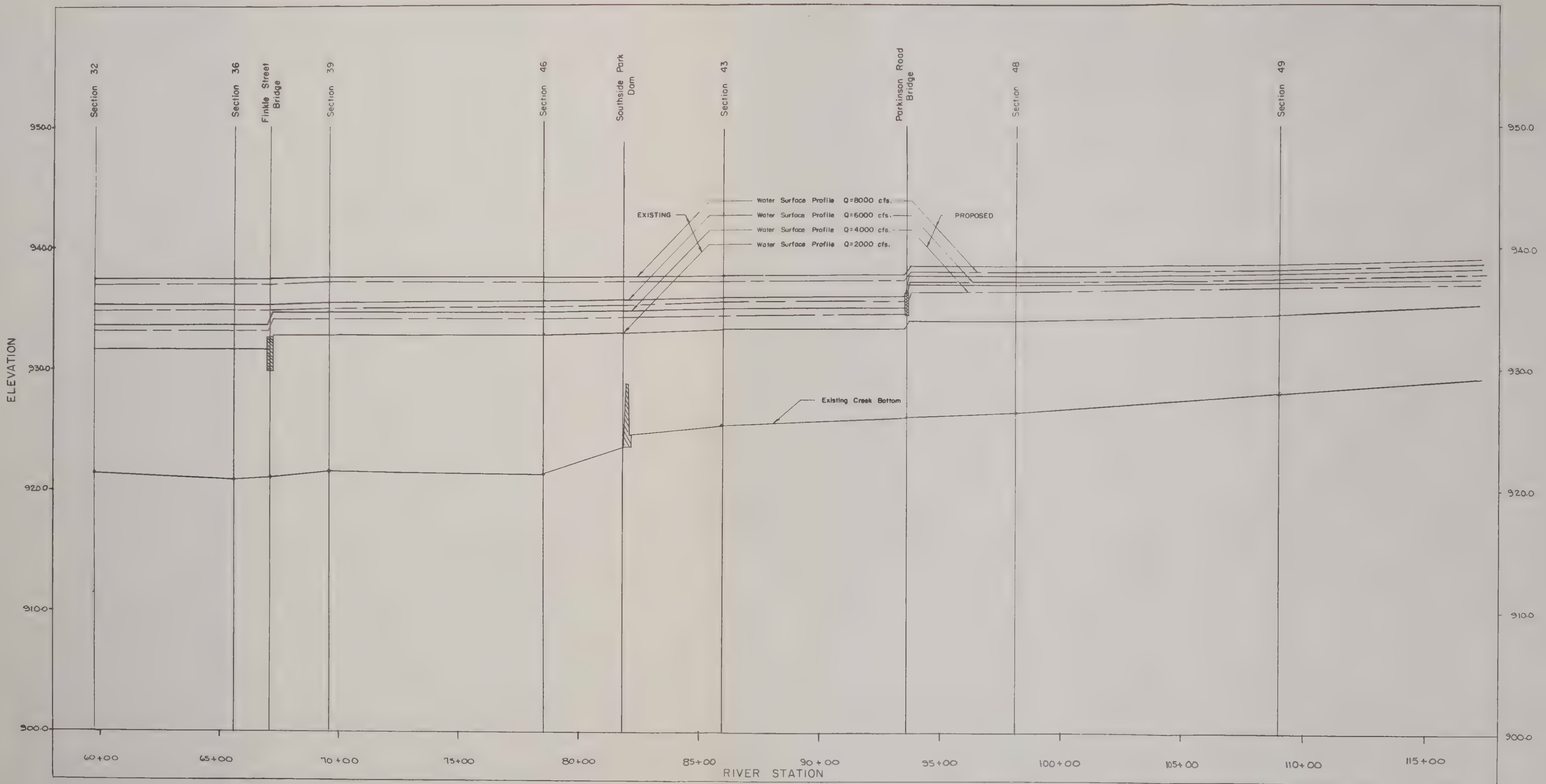
EXISTING AND PROPOSED CONDITIONS
CEDAR CREEK

SCALE: HOR. 1"=400' VERT. 1"=8'

DRAWING NO. 36

DATE OCT 15, 1961

NO.	REVISION MADE	BY	DATE



PROFILE

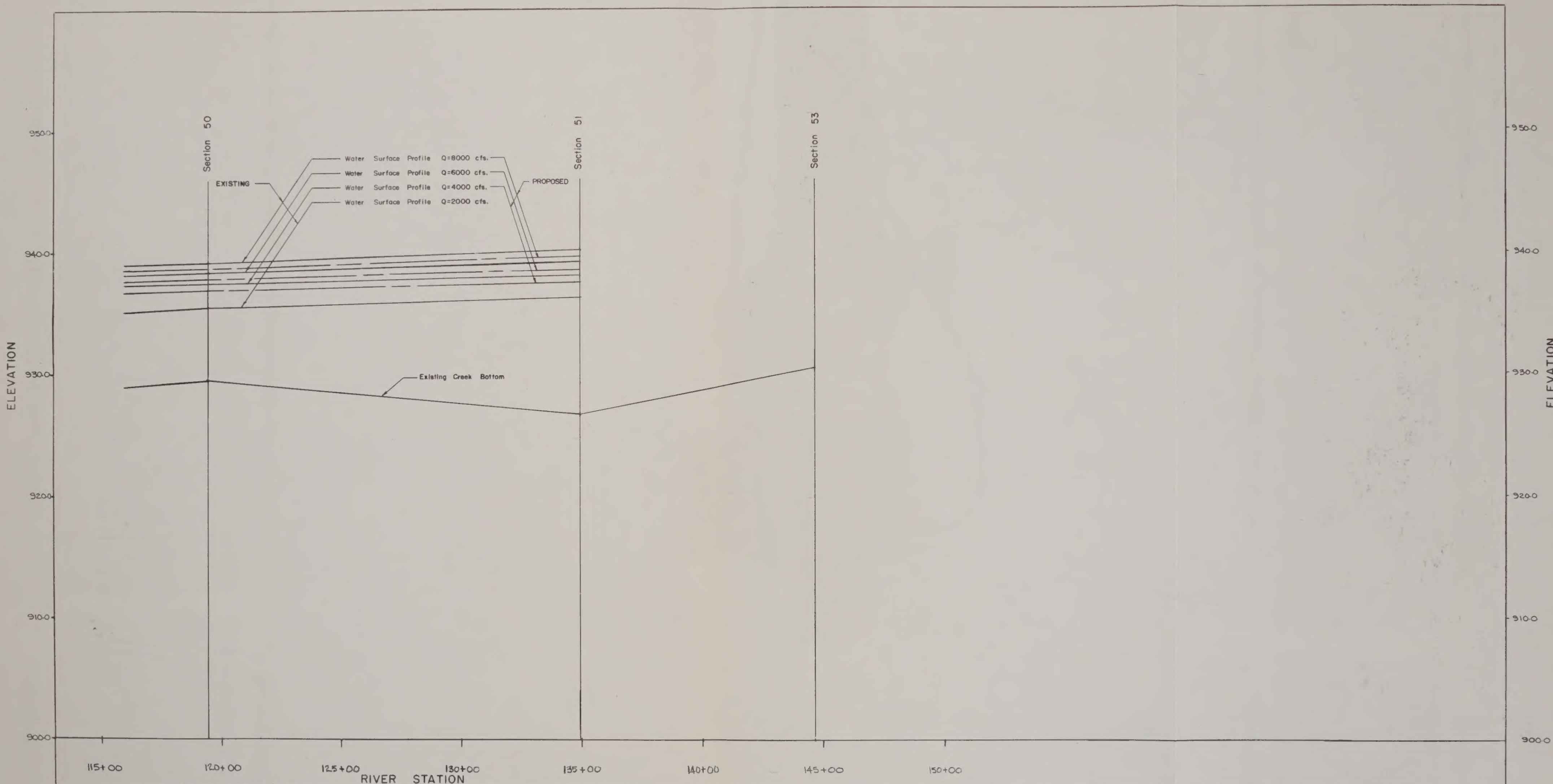
NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

COMPUTED WATER SURFACE PROFILES
EXISTING AND PROPOSED CONDITIONS
CEDAR CREEK

SCALE. HOR 1"=400' VERT. 1"=8'
DATE. OCT. 15, 1961

DRAWING NO. 37



PROFILE

NO.	REVISION MADE	BY	DATE

VANCE, NEEDLES, BERGENDOFF & SMITH LIMITED
CONSULTING ENGINEERS
WOODSTOCK, ONTARIO

COMPUTED WATER SURFACE PROFILES
EXISTING AND PROPOSED CONDITIONS
CEDAR CREEK

SCALE: HOR. 1"=400' VERT. 1"=8'
DATE: OCT. 15, 1981

DRAWING NO. 38